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# Freeway Capacity Analysis With Microscopic Simulation Model (FRESIM)

## (Phase II)

### Final Report

Prepared for

Research and Development
Turner-Fairbank Highway Research Center
Federal Highway Administration, U.S. Department of Transportation

Prepared by

Steven I-Jy Chien and Shoaib M. Chowdhury
New Jersey Institute of Technology
Institute for Transportation
Newark, NJ 07102

May, 1998

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# CHAPTER 1

## INTRODUCTION

The Highway Capacity Manual (HCM) and several traffic analysis computer packages (i.e., HCS, FREFLO, and FREQ, etc.) are being challenged with the problem of dealing with freeway capacity as input information. However, applying a microscopic simulation model, such as FRESIM (FREeway SIMulator) or CORSIM (CORridor SIMulator) to the problem, freeway capacity can be observed through performing a series simulations. In Phase I of this project, conducted by the Institute for Transportation at the New Jersey Institute of Technology, we concentrated on identifying factors affecting freeway capacity, indicating deficiencies, and recommending potential improvements in CORSIM for approximating freeway capacity. Recommendations and suggestions have been made in the final report (Phase I).

The major thrust of the efforts in Phase II will concentrate on (1) testing the factors (identified in Phase I) sensitivity to freeway capacity, (2) developing guidelines for approximating freeway capacity, and (3) analyzing parameters suggested by Highway Capacity Manual (1994-HCM) and results derived from the simulation model CORSIM.

## 1.1 Objectives and Scope

A freeway network is composed of three different types of components including the basic freeway section, the ramp junction and the weaving section. Besides unique operational characteristics of each freeway components, operational performance of each component is also affected by different geometric configurations, human driving behavior, and traffic characteristics. Although the 1994-HCM was developed based on an extensive database, gaps still exist. A calibrated microscopic simulation model, such as CORSIM, can be used to fill the gaps in the database. The objectives of this research are to (1) test the factors sensitive to freeway capacity by using the CORSIM model, (2) develop guidelines for approximating freeway capacity, and (3) compare results obtained from the CORSIM model and the 1994-HCM.

## 1.2 Report Organization

This report is organized into five chapters. Chapter 1 identifies the project objectives and scope, tests simulation models, and describes the methodology used for analyzing freeway capacity. The vast simulations are performed and results are analyzed in Chapters 2 through 4 where guidelines for approximating capacities for basic freeway sections, ramp junctions, and weaving areas are developed. Finally, conclusion, and further research are discussed in Chapter 5.

## 1.3 CORSIMs in TSIS 4.01 and 4.02

In accomplishing the objectives mentioned in the previous section, we performed evolutionary simulations by using two different CORSIM models embedded in TSIS 4.01 and 4.02. The CORSIM model in TSIS 4.01 is a released version from McTrans at University of Florida. Due to unreasonable results reported by us and other CORSIM users, FHWA endeavored lots of efforts to fix the identified problems, develop an enhanced CORSIM, and establish it into TSIS 4.02. In order to confidently use CORSIM in this project, we spent almost one month in testing CORSIM in TSIS 4.02 and compared results derived from CORSIMs in both TSIS 4.01 and 4.02. This test mainly focused on analyzing the impact of the minimum separation for generation of vehicles, specified in card type 70, to the maximum flow passing through a designated point. During the testing period, we had important findings that will be discussed in section 1.4 of this chapter.

## 1.4 Minimum Separation for Generation of Vehicles (MSGV)

Freeway capacity was approximated by using CORSIMs in both TSIS 4.01 and 4.02 through altering the value of the minimum separation time for generation of vehicles. For each pair of runs (every input file was simulated by CORSIMs in TSIS 4.01 and TSIS 4.02), the minimum separation time for generation of vehicles in both CORSIM models were kept the same (default MSGV = 2 seconds). MSGV governs the maximum

flow rate that vehicles can be generated by CORSIM and emitted onto the analyzed network from entry links. Based on the default MSGV of 2 seconds, the maximum entry flow is theoretically 1800 (3600 seconds divided by 2 seconds per vehicle) vehicles per hour per lane (vphpl), because the vehicle generation logic in CORSIM disallows the headway between an emitted vehicle and its leader greater than the MSGV.

Different MSGVs (e.g., one and two seconds) enable CORSIM to generate different flows discharged from entry links (e.g., theoritically 3600 and 1800 vphpl). Hence, if the entry flow is less than 1800 vphpl, the two-second MSGV is adequate to be selected for generating the anticipated entry flow. Otherwise, the MSGV is suggested to be one second instead of two seconds. For light entry flow Q, such as $Q < 1800$ vhp, the impact of 1-second and 2-second MSGV to the simulation results should be further tested. Simulation results should be very close for both cases.

The analyzed one-lane and two-lane basic freeway sections are shown in Figures 1-1 and 1-2. Each of both networks consists of three links with a relative long link (e.g., 10,000 ft) immediately after the entry link. The long link is designed to ensure proper randomness of vehicle arrivals at the beginning of the analyzed link without any bias caused by the pre-specified MSGV. Therefore, the capacities as well as the maximum flow passing through link (2,3) can be observed from the simulation results.

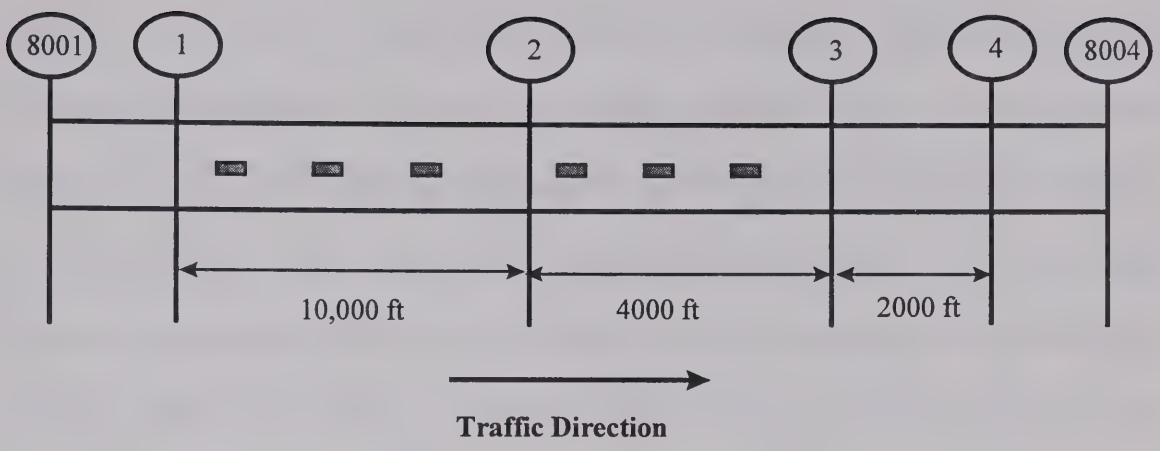



**Figure 1-1 Configuration of Basic Freeway Section (1-Lane Case)**

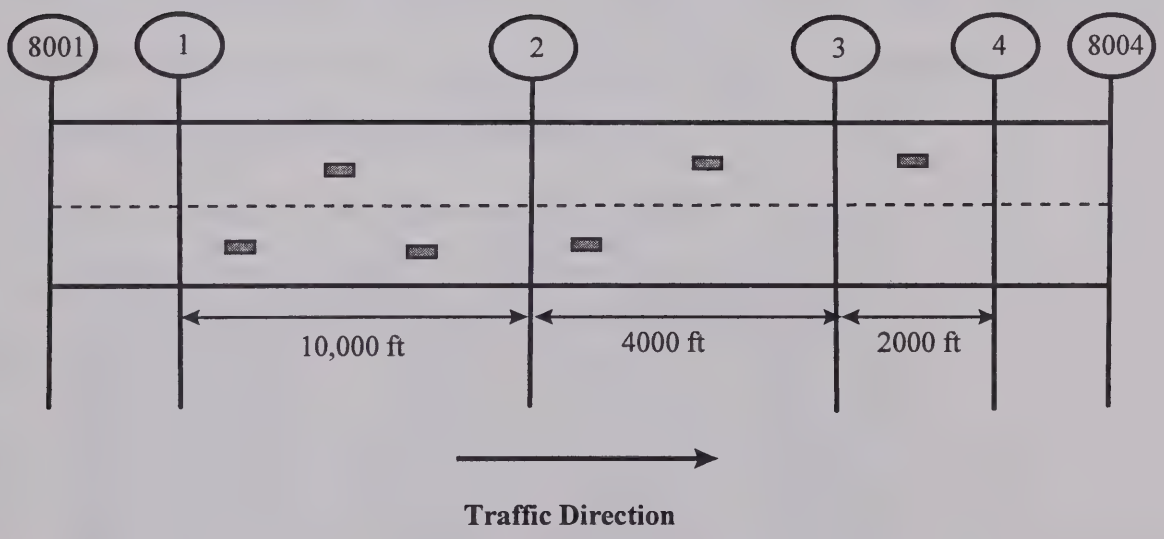



**Figure 1-2 Configuration of Basic Freeway Section (2-Lane Case)**

Through a series simulation runs and comparisons with different MSGVs (one and two seconds), the maximum flows are summarized in Table 1-1 and shown in Figures 1-3 and 1-4 (as MSGV = 1 second). Those figures demonstrated the relationship between the entry flow of link (8001,1) and the flow passing through link (2,3). We found that the maximum observed flows per lane per hour (using TSIS 4.02) on one-lane and two-lane cases are very close (e.g., 2338 and 2340 vphpl, respectively). However, by simulating the same network using TSIS 4.01, the maximum observed flows are relatively high (e.g., 2575 and 2459 vphpl for one-lane and two lane cases, respectively). For a two-lane case, the capacity per lane is lower than that in a one-lane case. This is due to the presence of discretionary lane change, with more aggressive drivers attempting to change their lane to gain advantage in position. This analysis presents the inconsistency between the results derived from CORSIM (TSIS4.01) and suggested by HCM, which addresses the important needs for properly modeling and calibrating the lane changing behavior in CORSIM where capacity is not a direct user input.

**Table 1-1 Entry Flow vs. Observed Flow ( MSGV = 1 second)**

| Entry flow | 1-Lane Case | | 2-Lane Case | |
|---|---|---|---|---|
| (vphpl) | TSIS 4.01 | TSIS 4.02 | TSIS 4.01 | TSIS 4.02 |
| 1000 | 998 | 999 | 999 | 1000 |
| 1200 | 1198 | 1200 | 1199 | 1202 |
| 1400 | 1399 | 1399 | 1400 | 1398 |
| 1600 | 1598 | 1598 | 1596 | 1597 |
| 1800 | 1801 | 1800 | 1799 | 1799 |
| 2000 | 2000 | 2001 | 1999 | 2000 |
| 2200 | 2198 | 2200 | 2199 | 2200 |
| 2400 | 2401 | 2338 | 2399 | 2340 |
| 2600 | 2575 | 2338 | 2459 | 2340 |
| 2800 | 2575 | 2338 | 2459 | 2340 |
| 3000 | 2575 | 2338 | 2459 | 2340 |

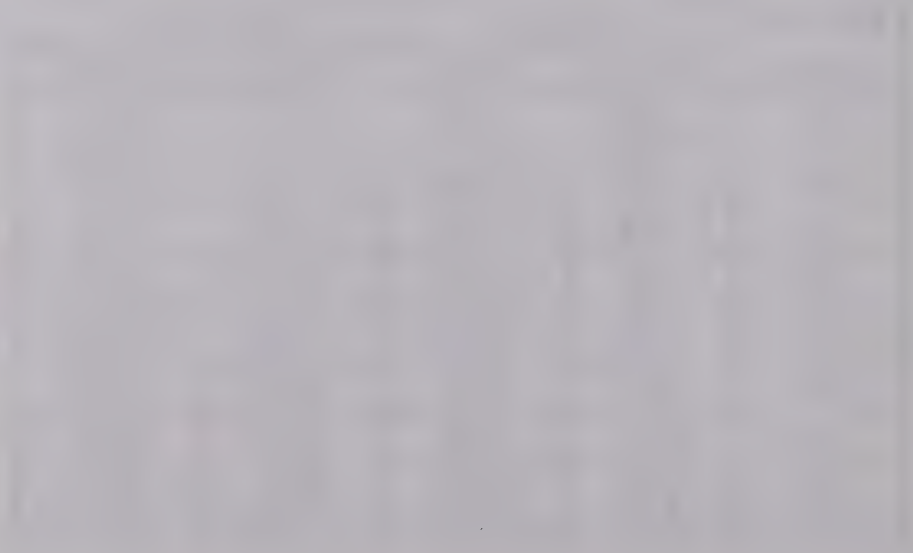


**Figure 1-3 Observed Flow vs. Entry Flow on Basic Freeway Section (MSGV = 1 second, 1-Lane Case)**

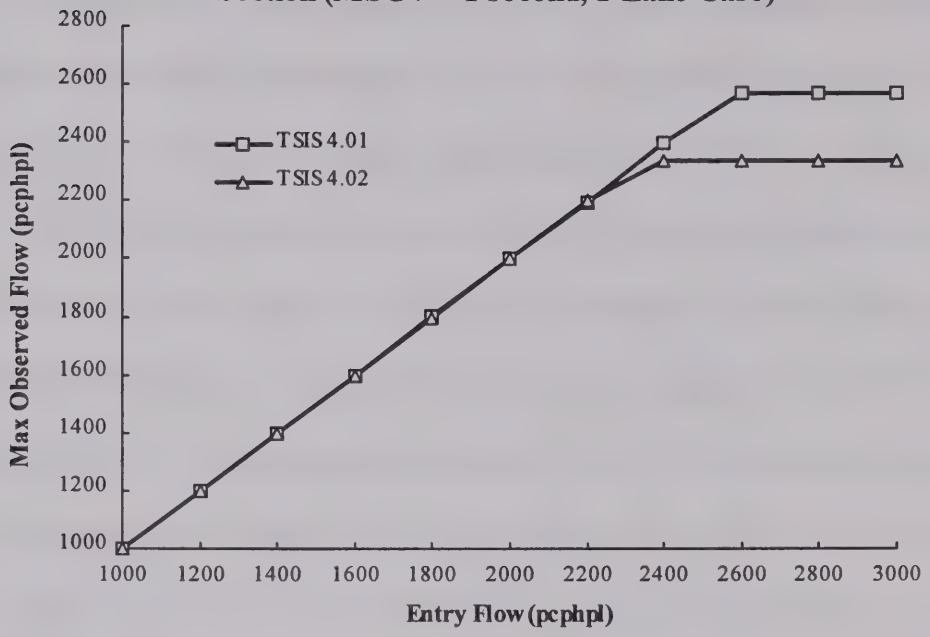
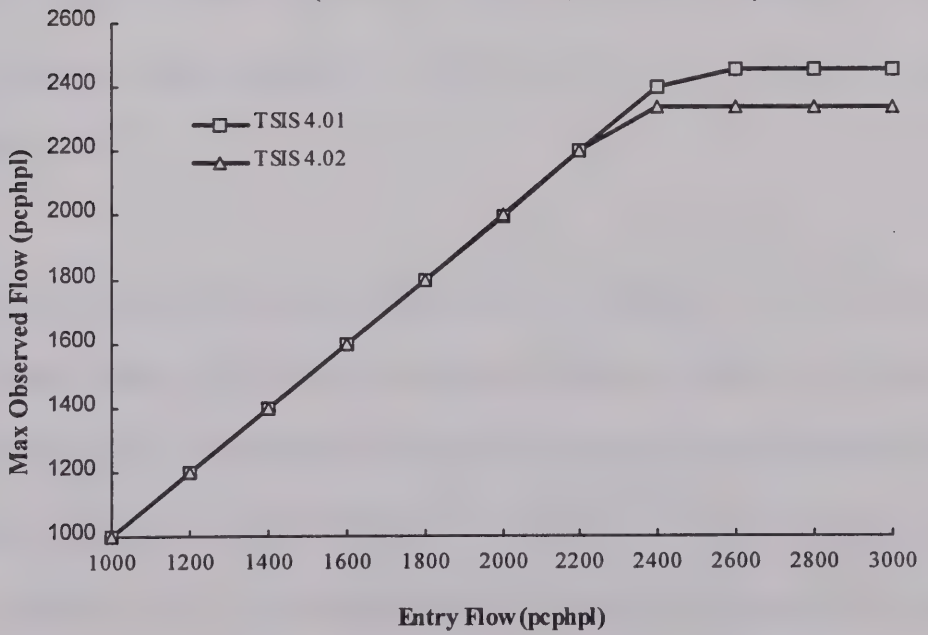



**Figure 1-4 Observed Flow vs. Entry Flow on Basic Freeway Section (MSGV = 1 second, 2-Lane Case)**



Figures 1-5 and 1-6 are generated based on information shown in Table 1-2 (MSGV=2 seconds) and present the relations again between the entry flow and the maximum observed flow passing through link (2,3). We found that the maximum observed flow generated by TSIS 4.02 for both one-lane and two-lane freeway sections are still close (e.g., 1798 and 1800 vphpl) and very consistent with the anticipated entry flow rate (1800 vphpl because of 2-second MSGV). However, the maximum flows observed from the results generated by TSIS 4.01 are relatively high and different (e.g., 2583 vphpl and 2490 vphpl), comparing with the maximum entry flow rate (1800 vphpl) generated by MSGV for both one and two-lane cases. Therefore, the observed flows 2583 and 2490 vphpl, are just too high and inconsistent with the expected flows (because two-second of MSGV could not produce traffic flow exceeding 1800 vphpl). It is also worth to notice that the capacities for both one and two-lane basic freeway sections obtained from TSIS 4.01 are higher than that obtained from TSIS 4.02 (for both MSGVs are 1 and 2 seconds). But, the capacity impact from the discretionary lane changing can not be observed from the results generated by TSIS 4.02. This problem should be further investigated as well.

Previous studies [3, 4] already demonstrated that traffic flows served by freeways are usually greater than 1800 vphpl. According to the findings addressed in 1994-HCM, the freeway capacity of a four-lane (two lanes per direction) freeway section is 2200 vphpl, while it is approximately 2300 vphpl on a six-lane or eight-lane freeway under idealized conditions ( e.g., all passenger cars, 12 feet lane width, and level terrain ). Therefore, the maximum flow observed from the results generated by TSIS 4.02 are more

reliable. In addition, the consistency between the observed capacity and the specified MSGV gives us more confidence to perform further capacity analysis by using TSIS 4.02 than using TSIS 4.01.

**Table 1-2 Entry Flow vs. Observed Flow ( MSGV=2 second)**

| Entry Flow | 1-Lane Case | | 2-Lane Case | |
|---|---|---|---|---|
| (vphpl) | TSIS 4.01 | TSIS 4.02 | TSIS 4.01 | TSIS 4.02 |
| 1000 | 998 | 999 | 1000 | 999 |
| 1200 | 1198 | 1203 | 1200 | 1200 |
| 1400 | 1404 | 1399 | 1404 | 1400 |
| 1600 | 1599 | 1598 | 1602 | 1598 |
| 1800 | 1801 | 1805 | 1799 | 1799 |
| 2000 | 2003 | 1798 | 2000 | 1800 |
| 2200 | 2198 | 1798 | 2201 | 1800 |
| 2400 | 2401 | 1798 | 2400 | 1800 |
| 2600 | 2575 | 1798 | 2490 | 1800 |
| 2800 | 2583 | 1798 | 2488 | 1800 |

**Figure 1-5 Observed Flow vs. Entry Flow on Basic Freeway Section (MSGV = 2 second, 1-Lane Case)**

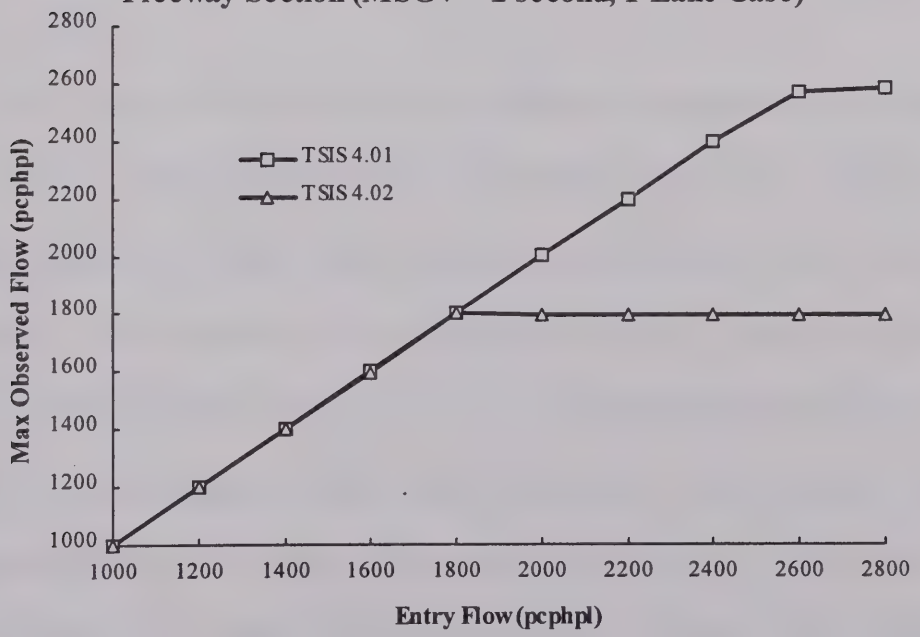
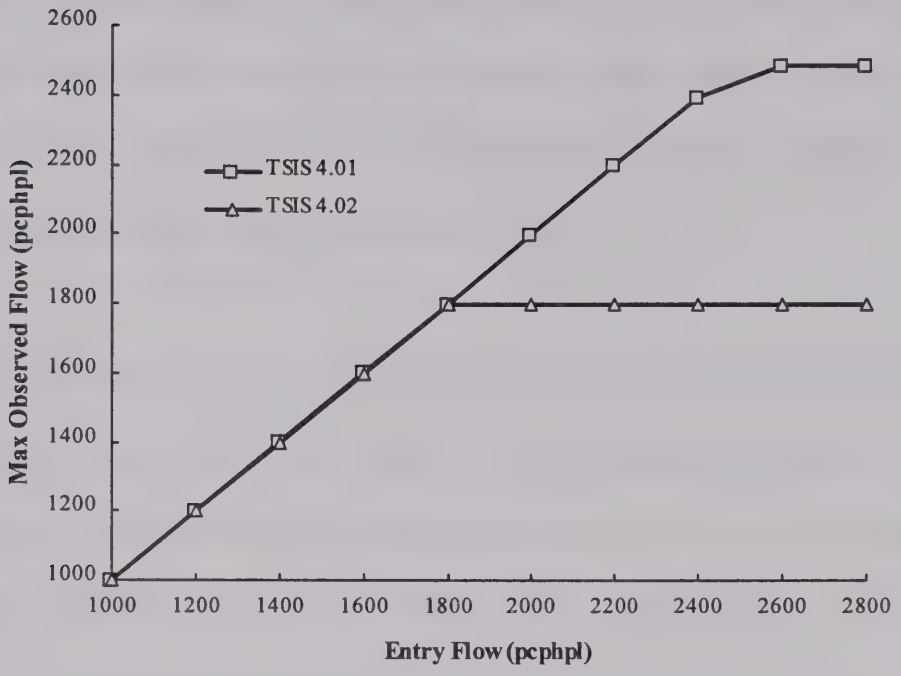


**Figure 1-6 Observed Flow vs. Entry Flow on Basic Freeway Section (MSGV = 2 second, 2-Lane Case)**




## 1.5 Procedure for Conducting Freeway Capacity Analysis

The aim of this study is to develop guidelines to approximate the capacity for various freeway configurations, traffic characteristics, and human driving behavior through simulations. The microscopic simulation model CORSIM in TSIS 4.02 is used to perform the analysis. According to the definition of the term "capacity" in 1994 HCM, "*capacity is the equivalent hourly flow rate of a maximum fifteen-minute flow*". Since CORSIM is a microscopic simulation model and is able to show the exact number of vehicles passing through a designated link during a time period, we define the maximum observed hourly flow passing through that link as "capacity". For approximating capacity, the entry flow is increased gradually in order to observe the maximum flow

passing through the designated link. The maximum flow of the designated link is achieved when the entry flow exceeds the observed flow. Thus, the link is not able to discharge any additional flow at that situation. Due to the expected statistic errors, the maximum observed flow varies, while the random number seed is changed. Hence, we treat the average (based on 10 one-hour simulation runs) maximum discharged flow rates as the capacity in order to reduce the statistic variance.

Since vehicles generated by CORSIM will be uniformly emitted onto lanes of each entry link (if the vehicle lane distribution is not specified), we design a longer link right after the entry link for improving the randomness of vehicle arrivals before entering the analyzed link. Guidelines for determining a proper link length for this purpose is not proceeded because of time constraint. However, a 10,000 feet link before the analyzed link is used, and it provides a natural vehicle arrival distribution for the analysis in this project.

# CHAPTER 2

# BASIC FREEWAY SECTIONS

The basic freeway section defined in HCM *is "Sections of the freeway that are unaffected either by merging or diverging movements at nearby ramps or by weaving movements".* This chapter discusses the capacity analysis for various basic freeway sections and is organized as follows:

- Factors sensitive to capacity (FSC).
- Experimental Design and Values of FSC
- Freeway Capacity and CFSF
- Freeway Capacity and Trucks
- Cross Impact of Grade Sections and Truck Percentages
- Equivalent Passenger Cars per Truck

## 2.1 Factors Sensitive to Capacity (FSC)

In the development of guidelines for estimating capacities of basic freeway sections, factors sensitive to capacity identified in Phase I are selected for further analysis and categorized into two groups based on their characteristics. Factors, categorized in

Type I, include the parameters related to driving behavior which can not be obtained from the field data, such as the car-following sensitivity factor, the driver reaction interval, and the hiatus period. These factors need to be well calibrated before performing simulation analysis. The values of the factors in Type II can be obtained from the field investigation, such as the geometric condition, the grade percentage, the lane width, and the truck percentage. Considering all factors sensitive to capacity as identified in Phase I, we found that only few factors significantly sensitive to capacities on basic freeway sections, including the car following sensitivity factor (Type I), the grade percentage (Type II), and the truck percentage (Type II).

Three factors are considered to estimate the capacity for the basic freeway section in 1994-HCM, including the lane width factor ($f_w$), driver population factor ($f_p$), and the heavy vehicle factor ($f_h$). Factors sensitive to freeway capacity in CORSIM are basically the same as them identified in HCM except the lane width factor. For example, the CFSF defined in CORSIM has the similar impact to capacity like the driver population factor defined in HCM. The impact of the combination of grade and truck percentages to capacity can be used to compare the heavy vehicle factor addressed in HCM as well. The lane width factor is not considered in this study because of the limitation in CORSIM. However, the narrow lane width impact to capacity may be simulated by specifying a fake rubbernecking factor for these narrow lanes but needs more testing on this issue.

## 2.2 Experimental Design and Values of FSC

For analyzing capacity of basic freeway sections, three basic freeway network (i.e., one-lane, two-lane, and three-lane cases) are designed in this chapter. Although most freeways have multiple lanes per direction in reality, we still analyze capacity of one-lane freeway. It may help to exclude the impact from lane changing behavior. As shown in Figures 1-1 and 1-2 (in Chapter 1), each of the three basic freeway sections consists of 5 links including one entry link, three internal links, and one exit link. The maximum number of vehicles discharged from link (2,3) at various conditions are observed while considering the impact of CFSF, truck population, and the grade percentage on freeway capacity.

## 2.3 Freeway Capacity and CFSF

In CORSIM, the default car-following sensitivity factor (CFSF) ranges between 6 and 15 tenths of a second and linearly increases from driver type 1 to 10. The value of CFSF will be assigned to each new generated vehicle once the driver type of the vehicle is determined through a random process. In addition, the CFSF values of drivers are identical for whom belonging to the same driver type. Unless a rubbernecking factor is specified when vehicles are passing through an incident zone, the CFSF of each driver is always fixed.

Simulations are conducted by analyzing various sets of car following sensitivity factors in different scenarios. In Scenario 1 (containing five cases), as shown in Table 2-1, we analyzed capacity while considering an identical car following sensitivity factor (altering the CFSFs from 6 to 14 tenths of a second) for all driver types. Therefore, the standard deviations of CFSF for all cases are zeros. Through this analysis, we found the impact of the CFSF value to the freeway capacity, while excluding the variance impact of CFSF to FSC (see Figure 2-1). In Scenario 2 (including twelve cases), as shown in Table 2-2, the CFSF values are designed to follow a distribution over ten driver types with various standard deviations (STD) of CFSF. These experiments are designed based on that the very cautious and aggressive drivers occupy only a small fraction of the total driver population. Hence, the impact of the standard deviation of the CFSF on freeway capacity could observed.

The relationship, summarized in Tables 2-3 through 2-6, between capacity and CFSF are found (one-lane through three-lane basic freeway sections) and shown in Figures 2-1 and 2-2. In Figure 2-1, the freeway capacity with zero standard deviation (STD) of the CFSF is presented. The well calibrated CFSF can represent driving behavior for the analyzed network and achieve the anticipated maximum observed flow discharged by links. The figure shows that while the CFSF increases from 6 to 14 tenths of a second, the observed capacity decreases almost 22% because of the increasing gap between pairs of vehicles. Additionally, capacities are very close for the basic freeway sections with one through three lanes on the mainline and is approximately 2340 pcphpl with the default CFSF. However, the relationship of the capacity and the number of lanes addressed in

HCM (the capacity is higher if the number of lanes is more) could hardly be found from the simulation results. These results are also different from the findings addressed by Halati, Lieu and Walker [6] (the equivalent capacity per lane of the multi-lane freeway is lower than that of the single lane freeway).

**Table 2-1 Scenario 1 (Identical CFSF Values for Ten Driver Types)**

| Case # | Driver Type | | | | | | | | | | Mean** | STD* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | | |
| 1 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 0 |
| 2 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 0 |
| 3 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 0 |
| 4 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 0 |
| 5 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 14 | 0 |

*STD=Standard Deviation (tenths of a second)

**Mean CFSF Value (tenths of a second)

**Table 2-2 Scenario 2 (CFSF with Common Mean and Various STDs)**

| Case # | Driver Type | | | | | | | | | | Mean | STD |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | | |
| 1 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 8 | 0 |
| 2 | 6 | 7 | 8 | 8 | 8 | 8 | 8 | 8 | 9 | 10 | 8 | 1 |
| 3 | 5 | 6 | 7 | 8 | 8 | 8 | 9 | 10 | 11 | 11 | 8 | 2 |
| 4 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 8 | 3 |
| 5 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 10 | 0 |
| 6 | 8 | 9 | 10 | 10 | 10 | 10 | 10 | 10 | 11 | 12 | 10 | 1 |
| 7 | 7 | 8 | 9 | 10 | 10 | 10 | 11 | 12 | 13 | 13 | 10 | 2 |
| 8 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 10 | 3 |
| 9 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 0 |
| 10 | 10 | 11 | 12 | 12 | 12 | 12 | 12 | 12 | 13 | 14 | 12 | 1 |
| 11 | 9 | 10 | 11 | 12 | 12 | 12 | 13 | 14 | 15 | 15 | 12 | 2 |
| 12 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 12 | 3 |

**Table 2-3 Capacity for Zero STD with Various CFSFs**

| Mean CFSF (tenths of a second) | Capacity (***pcphpl) | | |
|---|---|---|---|
| | 1-Lane Case | 2-Lane Case | 3-Lane Case |
| 6 | *2648 (**2.91) | 2639 (2.94) | 2635 (2.69) |
| 8 | 2566 (3.14) | 2565 (1.79) | 2565 (1.54) |
| 10 | 2418 (1.21) | 2418 (1.31) | 2418 (0.87) |
| 12 | 2328 (0.69) | 2328 (0.47) | 2328 (0.38) |
| 14 | 2066 (0.52) | 2065 (0.45) | 2065 (0.37) |

*Mean Capacity

**STD (Standard Deviation)

***pcphpl (Passenger car per hour per lane)

**Table 2-4 Capacity for Various STDs of CFSFs (1-Lane Case)**

| STD (tenths of a second) | Capacity (pcphpl) | | |
|---|---|---|---|
| | CFSF=0.8 sec. | CFSF=1.0 sec. | CFSF=1.2 sec. |
| 0 | 2566 (3.14) | 2418 (1.21) | 2328 (0.69) |
| 1 | 2562 (2.68) | 2413 (3.00) | 2315 (2.12) |
| 2 | 2517 (3.14) | 2377 (2.64) | 2257 (4.64) |
| 3 | 2473 (6.33) | 2335 (12.1) | 2204 (7.57) |

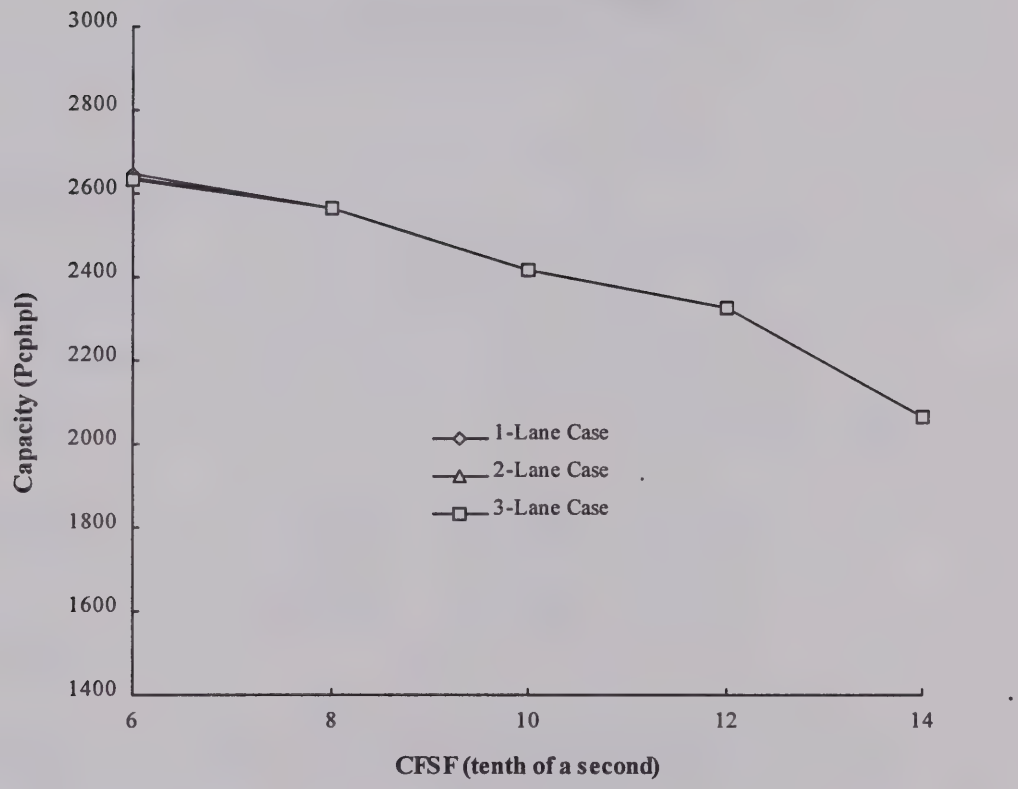
**Table 2-5 Capacity for Various STDs of CFSFs (2-Lane Case)**

| STD (tenths of a second) | Capacity (pcphpl) | | |
|---|---|---|---|
| | CFSF=0.8 sec. | CFSF=1.0 sec. | CFSF=1.2 sec. |
| 0 | 2565 (1.79) | 2418 (1.31) | 2328 (0.47) |
| 1 | 2561 (1.54) | 2413 (1.29) | 2316 (1.95) |
| 2 | 2516 (1.72) | 2376 (1.98) | 2258 (2.96) |
| 3 | 2476 (3.72) | 2335 (3.99) | 2206 (3.26) |

**Table 2-6 Capacity for Various STDs of CFSFs (3-Lane Case)**

| STD (tenths of a second) | Capacity (pcphpl) | | |
|---|---|---|---|
| | CFSF=0.8 sec. | CFSF=1.0 sec. | CFSF=1.2 sec. |
| 0 | 2565 (1.54) | 2418 (0.87) | 2328 (0.37) |
| 1 | 2561 (1.82) | 2413 (1.66) | 2316 (1.10) |
| 2 | 2517 (2.12) | 2377 (2.12) | 2257 (1.79) |
| 3 | 2476 (1.88) | 2336 (1.66) | 2203 (5.28) |

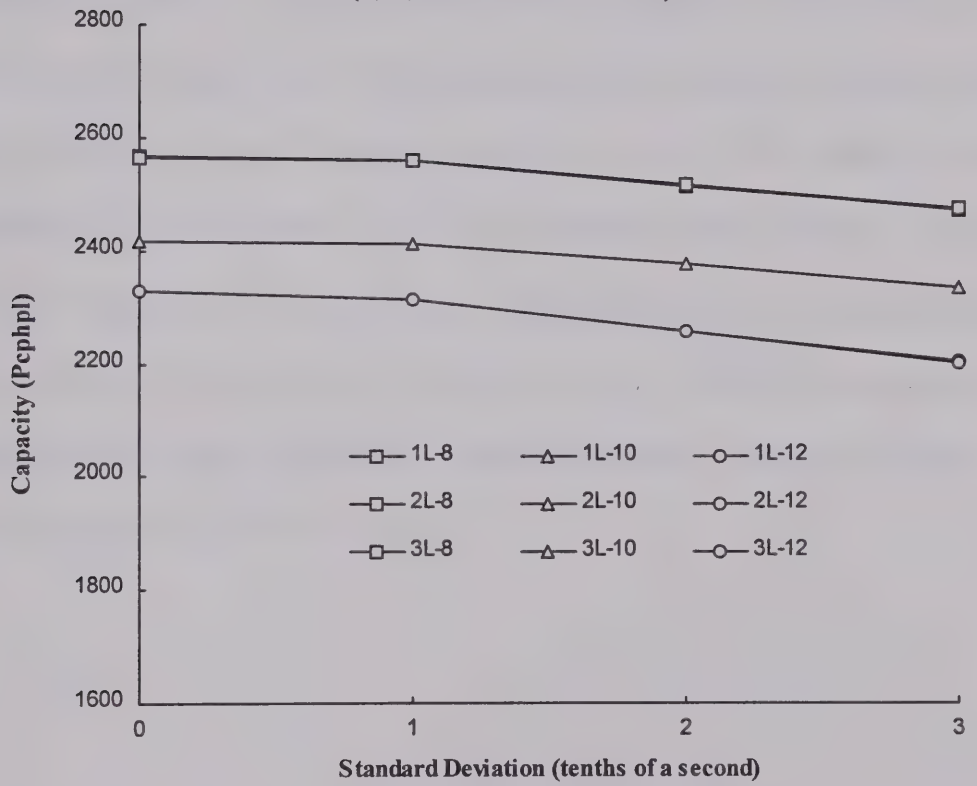
**Figure 2-1 Capacity vs. CFSF with Zero STD ( 0% Grade, All Passenger Cars)**




Taking into consideration of that the STD of the CFSF may affect freeway capacity, we analyze various cases for different means of CFSF (8, 10, and 12 tenths of a second) with various STDs (0, 1, 2, and 3 tenths of a second). The results are

summarized in Tables 2-4 through 2-6 and shown in Figure 2-2. Again, there is no significant difference observed in capacities for all 1, 2 and 3-lane cases. Figure 2-2 shows that the capacity decreases from 3% to 5% as the STD of the CFSF increases from 0 to 3 tenths of a second for all the cases. The capacity reduction rate is relatively small for lower STD of CFSF (i.e., for STD of CFSF =1 tenth of a second ) comparing to the larger STD of CFSF (i.e., for STD = 2 and 3 tenths of a second).

**Figure 2-2 Capacity vs. STD of CFSFs (1, 2, and 3-Lane Cases)**




XL-Y indicates X number of lanes with the mean CFSF=Y tenths of a second

## 2.4 Freeway Capacity and Truck Population

The major contribution of this section is to analyze the traffic impact due to different truck volumes and to develop guidelines for approximating capacity. To identify the impact of various truck percentages to freeway capacity, a series simulation runs are performed by simulating various freeway sections while altering the percentage of trucks. The guideline developed from the simulation results will help to approximate the freeway capacity, while considering different truck percentages with the default CFSF. Results are summarized in Table 2-7 and shown in Figure 2-3a, which are obtained from averaging 10 observations for each case. We found that the freeway capacity decreases almost 9% as the percentage of truck increases from 0% to 20 %. The capacity is decreased approximately 55 vphpl with every 5% increases in truck volumes. The capacity reduction is mainly due to the performance characteristics (e.g., acceleration/deceleration rate, and jerk) of trucks. Figure 2-3b shows capacity variations among each of 10 observed hourly capacity and that the standard deviation of observed maximum flows increases as percentage of truck increases.

**Table 2-7 Capacity for Various Truck Percentages**

| Truck Percentage | Capacity (vphpl) | | |
|---|---|---|---|
| | 1-Lane Case | 2-Lane Case | 3-Lane Case |
| 0 | 2335 (12.11) | 2335 (3.99) | 2336 (1.65) |
| 5 | 2291 (5.65) | 2283 (10.69) | 2277 (7.66) |
| 10 | 2233 (6.47) | 2233 (6.01) | 2223 (8.86) |
| 15 | 2186 (6.77) | 2186 (7.3) | 2176 (10.14) |
| 20 | 2133 (11.92) | 2134 (15.36) | 2133 (7.21) |

**Figure 2-3a Capacity vs. Truck Percentage**

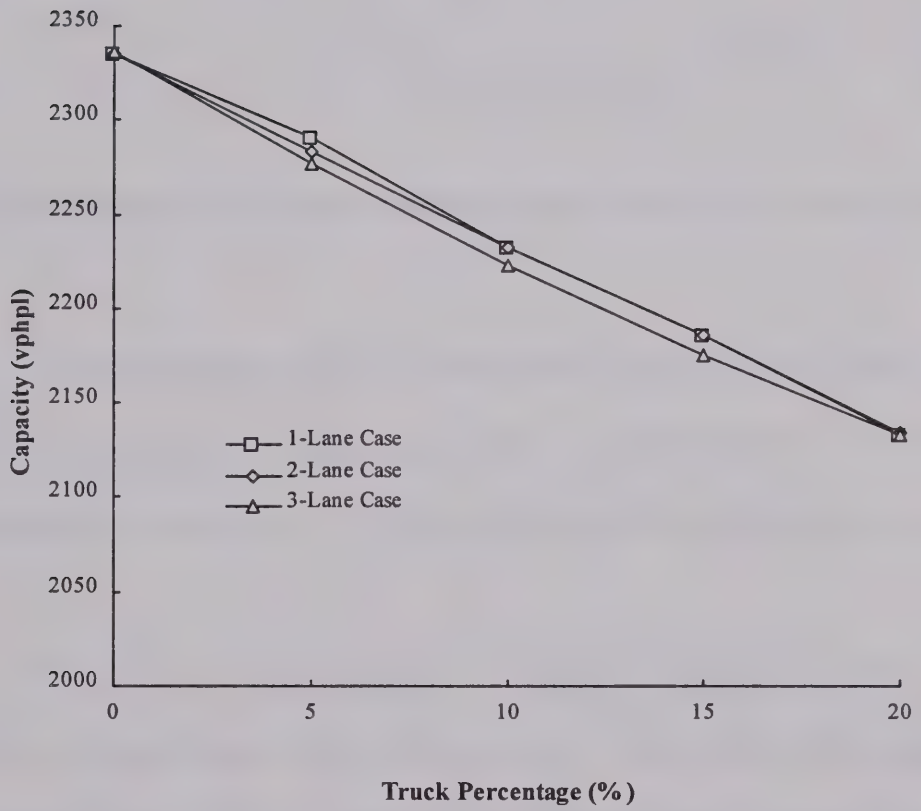
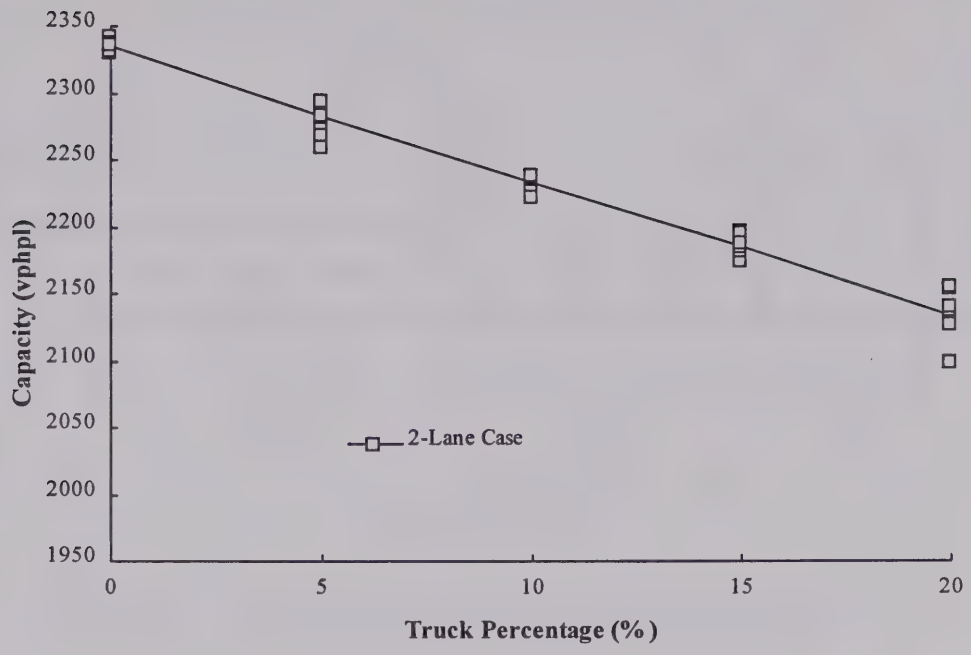


**Figure 2-3b Capacity vs. Truck Percentage (2-Lane Case)**




## 2.5 Cross Impact of Grade Sections and Trucks on Freeway Capacity

One of the important factors affecting freeway capacity is the grade section length and its slope. After simulating various cases, we found that the cross impact of the increasing truck population and the grade percentage further deteriorate the capacity.

The main task in this section is to explore the relationship between the capacity and the truck percentage, while grade percentages over various section lengths are also considered. Figures 2-4a and 2-4b show the analyzed freeway segment (one-lane basic freeway section) and its geometric profile, respectively. Variables, such as truck

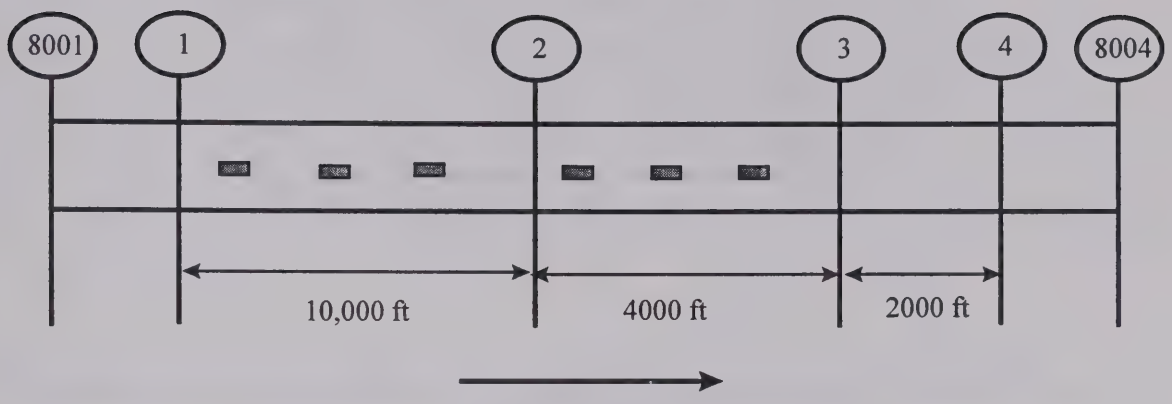



**Figure 2-4a Configuration of One-Lane Basic Freeway Section**

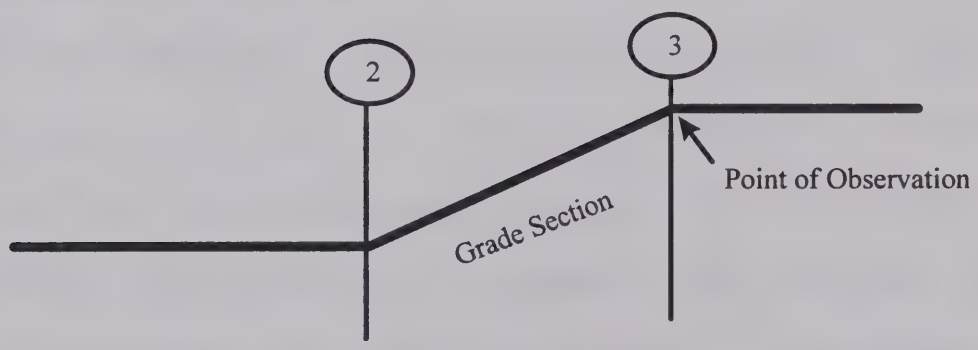



**Figure 2-4b Geometric Profile of the Analyzed Freeway Segment**

**Table 2-8 Variables Used for Cross Impact Capacity Analysis**

| Grade % | Truck % | *GSL (ft) |
|---|---|---|
| 0 | 0 | 1000 |
| 2 | 5 | 2000 |
| 4 | 10 | 3000 |
| 6 | 15 | 4000 |
| - | 20 | 5000 |
| - | - | 6000 |

*GSL (ft)= Grade Section Length

percentages, grades, and grade section lengths, and their values used in the analysis are summarized in Table 2-8.

The number of simulations performed in this section is 3600 (e.g., 3*4*5*6*10 = 3600), which covers simulations for 3 different basic freeway segments (1, 2, and 3 lanes), 4 different grade percentages (0%, 2%, 4% and 6%), 5 different truck percentages (0%, 5%, 10%, 15% and 20%), 6 different grade section lengths (1000, 2000, 3000, 4000, 5000, and 6000 ft.), and 10 observations (10 one-hour simulation runs). Results are summarized in Tables 2-9 through 2-17, while Figures 2-5 through 2-7 plot the maximum observed flows at various situations. It should be noted that these relations are derived based on the default CFSF, and the similar relations based on other sets of CFSF may be explored in future studies.

**Table 2-9 Capacity (vphpl) of 1-Lane Freeway Section with 2% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| **Section Length (ft)** | **0%** | **5%** | **10%** | **15%** | **20%** |
| **1000** | *2337 (**4.2) | 2282 (6.93) | 2237 (5.35) | 2186 (9.56) | 2145 (9.79) |
| **2000** | 2336 (3.71) | 2288 (9.33) | 2235.7 (9.12) | 2197 (11.65) | 2148 (8.55) |
| **3000** | 2333 (2.27) | 2286 (9.72) | 2243 (8.78) | 2197 (12.17) | 2152 (8.1) |
| **4000** | 2337 (5.82) | 2287 (10.11) | 2228 (37.08) | 2138.4 (31.85) | 2067 (28.97) |
| **5000** | 2337 (4.14) | 2287 (13.25) | 2176 (17.76) | 2117 (23.43) | 2045 (19.06) |
| **6000** | 2338 (5.28) | 2266 (21.56) | 2157 (17.48) | 2109 (34.38) | 2055 (15.51) |

*mean capacity (vphpl)
**STD (vphpl)

**Table 2-10 Capacity (vphpl) of 2-Lane Freeway Section with 2% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| **Section Length (ft)** | **0%** | **5%** | **10%** | **15%** | **20%** |
| **1000** | 2337(4.62) | 2282 (8.99) | 2236 (8.21) | 2186 (19.69) | 2140 (65.74) |
| **2000** | 2337 (3.32) | 2290 (8.11) | 2206 (28.48) | 2128 (18.42) | 2056 (18.9) |
| **3000** | 2337 (2.85) | 2275 (15.67) | 2155 (19.9) | 2070 (16.6) | 2017 (16.77) |
| **4000** | 2336 (20.45) | 2230 (22.8) | 2129 (26.25) | 2053 (19.8) | 2004 (11.4) |
| **5000** | 2337 (4.46) | 2224 (26.67) | 2092 (9.45) | 2043 (17.08) | 1971 (48.77) |
| **6000** | 2337 (2.86) | 2226 (19.47) | 2109 (19.13) | 2031 (10.86) | 1973 (11.24) |

**Table 2-11 Capacity (vphpl) of 3-Lane Freeway Section with 2% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| **Section Length (ft)** | **0%** | **5%** | **10%** | **15%** | **20%** |
| **1000** | 2336 (2.7) | 2281 (8.9) | 2234 (13) | 2181 (16.9) | 2123 (13) |
| **2000** | 2336 (2.8) | 2267 (6.9) | 2199 (16.6) | 2109 (15.6) | 2052 (7.8) |
| **3000** | 2338 (3.03) | 2265 (20.7) | 2149 (18.5) | 2071 (15.5) | 2013 (10.3) |
| **4000** | 2337 (1.3) | 2243 (9.3) | 2115 (18.53) | 2039 (22.8) | 1984 (15.7) |
| **5000** | 2337 (2.66) | 2218 (14.63) | 2107 (26.88) | 2036 (11.54) | 1964 (17.72) |
| **6000** | 2337 (3.42) | 2211 (17.82) | 2096 (14.45) | 2019 (17.09) | 1961 (11.35) |

**Table 2-12 Capacity of 1-Lane Freeway Section with 4% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| Section Length (ft) | 0% | 5% | 10% | 15% | 20% |
| 1000 | 2334 (4.73) | 2280 (17.94) | 2087 (25.51) | 1976 (44.98) | 1893 (31.24) |
| 2000 | 2337 (4.83) | 2109 (35.51) | 1963 (28.83) | 1855 (26.77) | 1788 (32.37) |
| 3000 | 2337 (4.26) | 2039 (36.03) | 1874 (40.73) | 1795 (27.24) | 1745 (30.72) |
| 4000 | 2336 (4.65) | 1988 (37.15) | 1847 (36.02) | 1764 (22.13) | 1705 (21.32) |
| 5000 | 2336 (5.29) | 1989 (65.25) | 1830 (31.60) | 1766 (30.39) | 1698 (19.90) |
| 6000 | 2337 (2.91) | 1947 (40.11) | 1813 (32.00) | 1752 (24.54) | 1684 (20.59) |

**Table 2-13 Capacity (vphpl) of 2-Lane Freeway Section with 4% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| Section Length (ft) | 0% | 5% | 10% | 15% | 20% |
| 1000 | 2336 (2.3) | 2219 (20.5) | 2042 (22.8) | 1925 (30.22) | 1840 (22.98) |
| 2000 | 2337 9 (3.6) | 2099 (23.96) | 1926 (48) | 1825 (27.2) | 1735 (17.75) |
| 3000 | 2336 (2.68) | 2018 (21.7) | 1866 (17.77) | 1772 (25.95) | 1703 (18.5) |
| 4000 | 2337 (3.64) | 2002 (29.10) | 1855 (27.2) | 1752 (20.22) | 1684 (23.4) |
| 5000 | 2335 (3.90) | 1952 (19.09) | 1822 (24.58) | 1737 (10.34) | 1672 (12.98) |
| 6000 | 2336 (4.21) | 1965 (12.92) | 1825 (21.05) | 1732 (19.34) | 1663 (19.00) |

**Table 2-14 Capacity (vphpl) of 3-Lane Freeway Section with 4% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| Section Length (ft) | 0% | 5% | 10% | 15% | 20% |
| 1000 | 2337 (3.6) | 2231 (17.3) | 2045 (15.2) | 1935 (26.1) | 1844 (12.5) |
| 2000 | 2337 (2.7) | 2094 (18) | 1931 (21.5) | 1821 (14.2) | 1740 (13.7) |
| 3000 | 2338 (3.6) | 2030 (23.2) | 1866 (24) | 1772 (14.5) | 1705 (18.8) |
| 4000 | 2337 (2.3) | 1996 (13.8) | 1854 (17.99) | 1749 (14.7) | 1676 (9.3) |
| 5000 | 2337 (3.00) | 1974 (11.99) | 1831 (20.41) | 1742 (18.57) | 1663 (8.93) |
| 6000 | 2338 (2.58) | 1969 (21.05) | 1821 (14.04) | 1724 (18.18) | 1663 (14.25) |

**Table 2-15 Capacity (vphpl) of 1-Lane Freeway Section with 6% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| Section Length (ft) | 0% | 5% | 10% | 15% | 20% |
| 1000 | 2334 (6.86) | 2084 (48.71) | 1816 (29.32) | 1664 (24.8) | 1550 (39.62) |
| 2000 | 2337 (5.89) | 1882 (38.5) | 1624 (51.21) | 1523 (36.75) | 1415 (30.16) |
| 3000 | 2335 (3.71) | 1736 (66.15) | 1552 (49.38) | 1476 ((39.9) | 1394 (46.53) |
| 4000 | 2337 (3.84) | 1677 (71.34) | 1511 (35.9) | 1425 (51.79) | 1345 (20.15) |
| 5000 | 2337 (4.03) | 1614 (45.85) | 1477 (35.31) | 1406 (37.23) | 1351 (18.17) |
| 6000 | 2334 (3.22) | 1622 (70.50) | 1441 (43.04) | 1385 (29.60) | 1339 (23.74) |

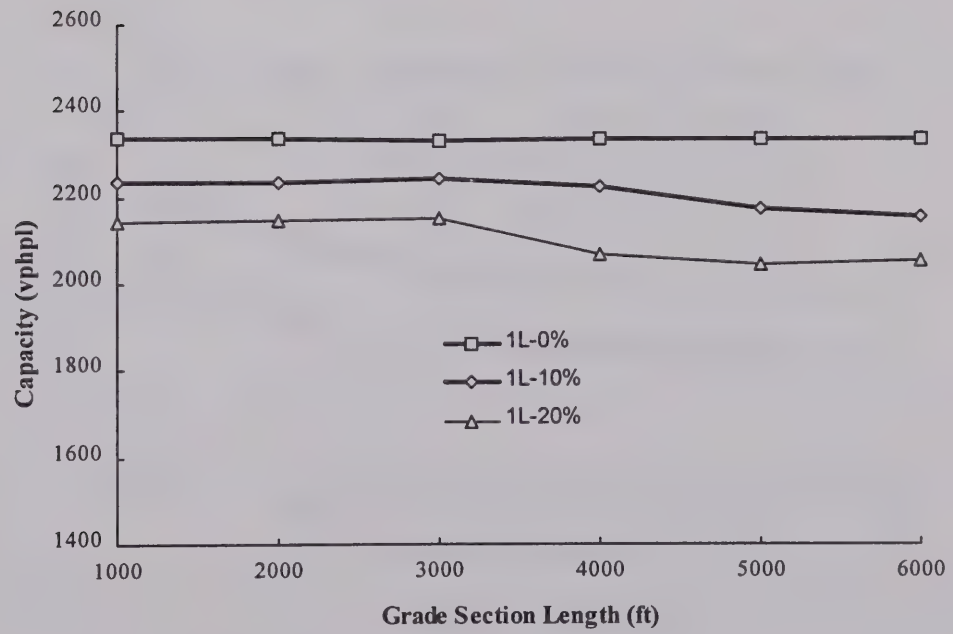
**Table 2-16 Capacity (vphpl) of 2-Lane Freeway Section with 6% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| Section Length (ft) | 0% | 5% | 10% | 15% | 20% |
| 1000 | 2336 (2.98) | 2052 (38.9) | 1792 (38.66) | 1657 (13.7) | 1544 (44.4) |
| 2000 | 2338 (3.3) | 1881 (38.2) | 1657 (31.8) | 1523 (31.85) | 1435 (25.4) |
| 3000 | 2338 9(3.2) | 1787 (32.47) | 1599 (22.32) | 1478 (20.2) | 1401 (21.3) |
| 4000 | 2337 (2) | 1734 (21.9) | 1554 (33.42) | 1460 (37.43) | 1376 (25.17) |
| 5000 | 2337 (2.56) | 1734 (31.55) | 1539 (31.62) | 1463 (27.42) | 1360 (19.44) |
| 6000 | 2339 (4.73) | 1682 (28.68) | 1545 (32.02) | 1452 (24.75) | 1361 (19.19) |

**Table 2-17 Capacity (vphpl) of 3-Lane Freeway Section with 6% Grade**

| | Truck Percentage | | | | |
|---|---|---|---|---|---|
| Section Length (ft) | 0% | 5% | 10% | 15% | 20% |
| 1000 | 2336 (2.3) | 2043 (35.9) | 1799 (64.7) | 1648 (23.7) | 1558 (35.1) |
| 2000 | 2336 (2.9) | 1875 (32.2) | 1642 (27.5) | 1540 (38.5) | 1435 (14.6) |
| 3000 | 2338 (2.97) | 1787 (28.9) | 1606 (18.7) | 1491 (30.1) | 1397 (17.98) |
| 4000 | 2336 (3.52) | 1746 (26.7) | 1589 (25.3) | 1467 (22.7) | 1393 (13.2) |
| 5000 | 2336 (3.10) | 1758 (24.68) | 1566 (31.43) | 1462 (19.49) | 1382 (27.55) |
| 6000 | 2336 (2.53) | 1732 (25.41) | 1554 (21.83) | 1442 (31.12) | 1373 (24.98) |

**Figure 2-5a Capacity vs. Grade Section Length with 2% Grade (1-Lane Cases with 0, 10, and 20 % Trucks)**




*L-** % means *lane with ** % truck

**Figure 2-5b Capacity vs. Grade Section Length with 2% Grade ( 2 and 3-Lane Cases with 0, 10, and 20 % Trucks)**

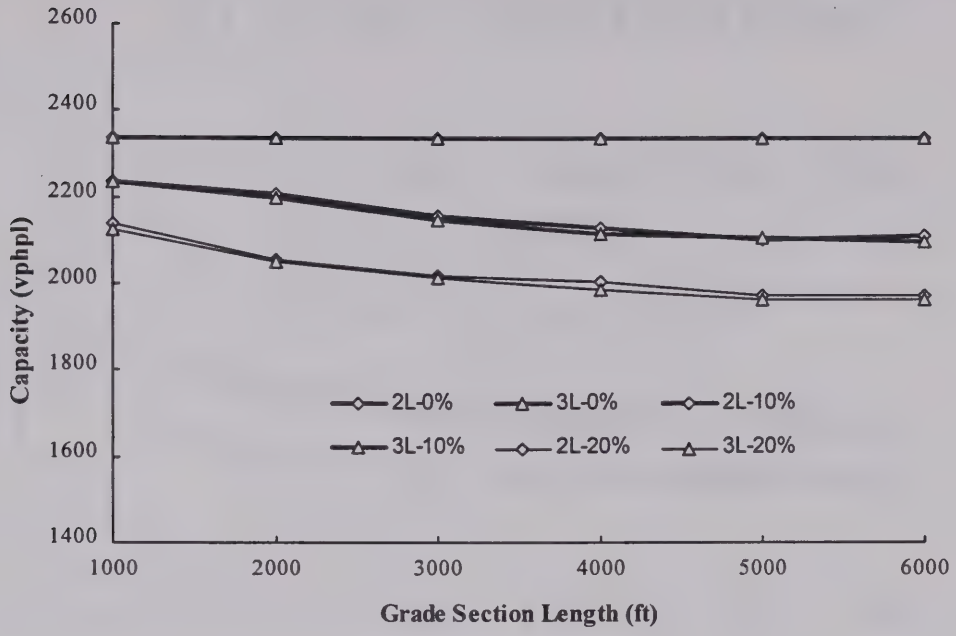



**Figure 2-6 Capacity vs. Grade Section Length with 4% Grade (1, 2, and 3-Lane Cases with 0, 10, and 20 % Trucks)**

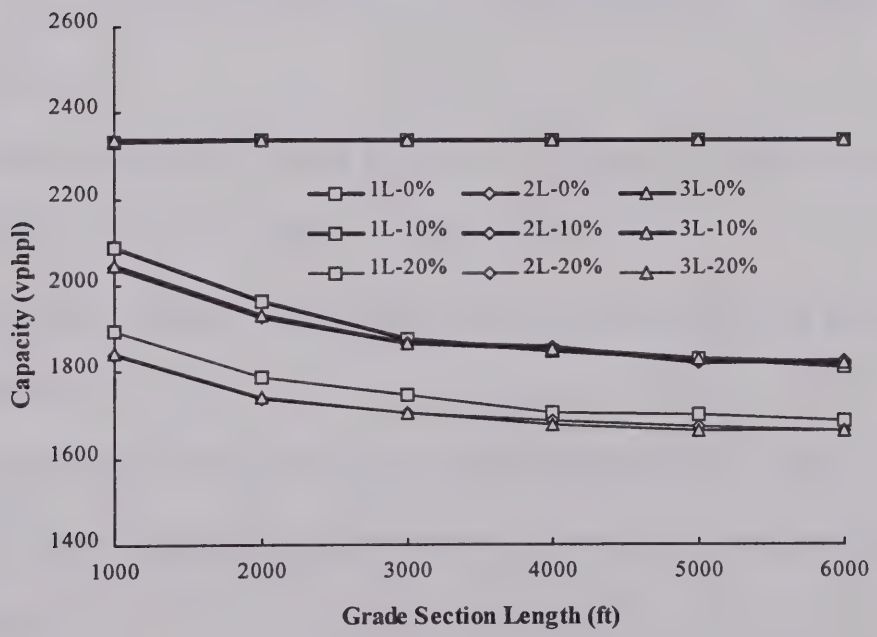
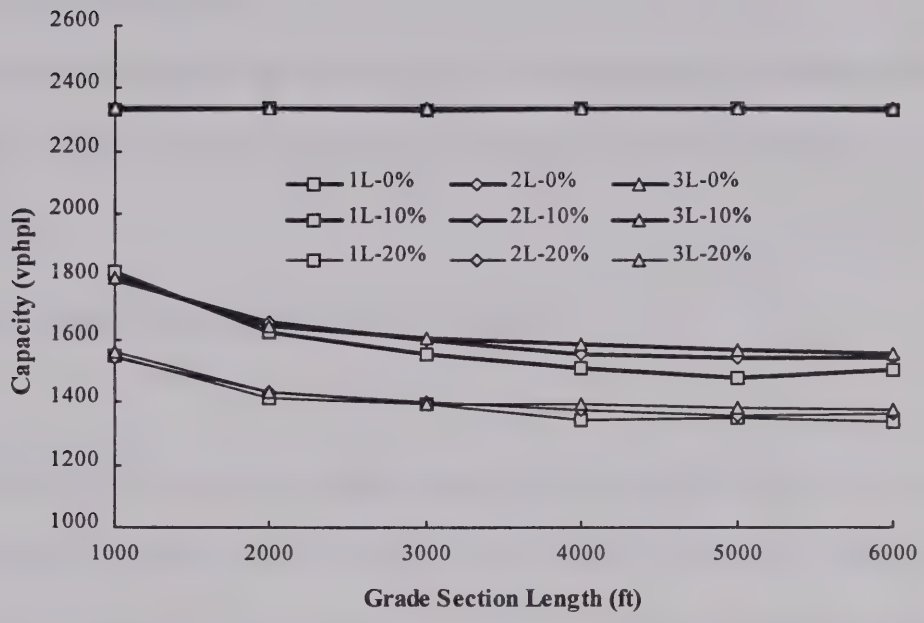


**Figure 2-7 Capacity vs. Grade Section Length with 6% Grade (1, 2, and 3-Lane Cases with 0, 10, and 20 % Trucks)**




Conclusions are made based on simulation results shown in Figures 2-5 through 2-7 and stated as follows:

1. The grade percentage (e.g., from 0 to 6% ) has no impact on basic freeway section capacity when there are no truck in the traffic stream.
2. Freeway capacity decreases and approaches to a constant when the grade section length increases.
3. The length of grade section that does not affect freeway capacity varies with both grade and truck percentage and decreases with increase of both grade and truck percentage.

4. When the truck percentage is fixed, if the grade percentage increases, the freeway capacity decreases (the capacity reduction rate per grade percentage increases) (See Table 2-9 through 2-17).
5. When the grade percentage is fixed, if the truck percentage increases, reduction of freeway capacity per percentage of truck decreases (See Table 2-9 through 2-17).

## 2.6 Equivalent Passenger Cars per Truck

Based on the figures and tables generated in the previous section, the number of passenger cars equivalent to a truck ($E_T$) is derived and summarized in Table 2-18. We developed $E_T$ for various section lengths on 2%, 4%, and 6 % grades with various truck volumes (e.g., 5%, 10%, 15% and 20%). This analysis can be extended to determine $E_T$ for any length of the grade section. In Table 2-18, the $E_T$ values presented in the parenthesis are obtained from HCM. Comparing $E_T$ values derived from CORSIM and HCM, we found that CORSIM and HCM suggest consistent values of $E_T$ only for 2 % grade cases. For 4% and 6% grade situations, these values are significantly inconsistent, especially when both the truck volume is large and the grade section length is long. It should be noted here that the $E_T$ values generated from CORSIM are based on default CFSF and truck fleet percentages. If CFSF or truck fleet percentage varies, $E_T$ may vary as well. This analysis demonstrates that once an analyzed freeway network has been well calibrated, the proper $E_T$ of the network can be derived instead of using the $E_T$ suggested by HCM. In addition, the $E_T$ values derived from the simulation results can be used for

various numbers of lanes on analyzed networks. For easily comparing $E_T$'s calculated from CORSIM results and suggested by HCM, the suggested $E_T$ from CORSIM results is obtained from the average capacity of two and three lane cases. This analysis also shows that HCM can still provide reliable $E_T$ when the grade percentage is low, such as 2%. However, if the grade percentage increases, it is suggested to use simulation approaches for determining the proper $E_T$ . Following equations, addressed in 1994-HCM, were used in calculating $E_T$ based on results obtained from CORSIM.

When the PHF (peak hour factor), the lane width ($f_W$), and the driver population ($f_P$) factors are all equal to 1, the capacity $V_C$ under ideal conditions is

$$V_C = \frac{V_P}{f_{HV}} \tag{2-1}$$

where $V_P$ is observed capacity (vphpl). According to HCM, the heavy vehicle factor can be obtained from Eq. 2-2 as

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} \tag{2-2}$$

where $f_{HV}$ is the heavy vehicle adjustment factor, and $P_T$ is the truck percentage. By substituting Eq. 2-2 into Eq. 2-1, Eq. 2-3 is derived and can be used to develop the adjusted $E_T$ Table for the analyzed network.

$$E_T = 1 + (\frac{V_C}{V_P} - 1)\frac{1}{P_T} \tag{2-3}$$

**Table 2-18 Equivalent Passenger Cars to One Truck on Specific Grades**

| Grade (%) | GSL(ft) | 5% truck | 10% truck | 15% truck | 20% truck |
|---|---|---|---|---|---|
| 2% | 1000 | 1.51(1.5) | 1.47(1.5) | 1.48 (1.5) | 1.49 (1.5) |
| | 2000 | 1.54 (1.5) | 1.62 (1.5) | 1.7 (1.5) | 1.69 (1.5) |
| | 3000 | 1.62(1.5) | 1.87 (1.5) | 1.87 (1.5) | 1.81 (1.5) |
| | 4000 | 1.92(2.0) | 2.03 (1.5) | 1.96 (1.5) | 1.87 (1.5) |
| | 5000 | 2.07 (2.0) | 2.14 (1.5) | 1.98 (1.5) | 1.95 (1.5) |
| | 6000 | 2.09 (3.0) | 2.13 (2.5) | 2.04 (2.0) | 1.95 (2.0) |
| 4% | 1000 | 2.03 (1.5) | 2.44 (1.5) | 2.42 (1.5) | 2.35 (1.5) |
| | 2000 | 3.32 (4.0) | 3.13 (3.0) | 2.89 (3.0) | 2.73 (2.5) |
| | 3000 | 4.12 (6.5) | 3.54 (5.0) | 3.14 (4.5) | 2.87 (4.0) |
| | 4000 | 4.41 (7.0) | 3.61 (5.5) | 3.24 (5.0) | 2.96 (4.5) |
| | 5000 | 4.84 (7.0) | 3.81 (5.5) | 3.3 (5.0) | 3.02 (4.5) |
| | 6000 | 4.79 (7.5) | 3.84 (6.0) | 3.36 (5.0) | 3.03 (5.0) |
| 6% | 1000 | 3.85 (3.0) | 4.03 (2.5) | 3.77 (2.5) | 3.54 (2.0) |
| | 2000 | 5.92 (8.5) | 5.18 (6.5) | 4.52 (6.0) | 4.15 (6.0) |
| | 3000 | 7.19 (10) | 5.6 (8.0) | 4.84 (8.0) | 4.36 (7.5) |
| | 4000 | 7.89 (10) | 5.89 (8.5) | 4.99 (8.0) | 4.45 (7.5) |
| | 5000 | 7.8 (10) | 6.07 (8.5) | 5 (8.0) | 4.53 (7.5) |
| | 6000 | 8.42 (10.0) | 6.1 (8.5) | 5.11 (8.0) | 4.56 (7.5) |

# CHAPTER 3

## FREEWAY RAMP JUNCTIONS

The freeway ramp junction is defined in HCM as *"The roadway area over which an on-ramp or off-ramp joins the mainline of a freeway. In the merge area, individual on-ramp vehicles attempt to find gaps in the adjacent freeway lane traffic stream. At off-ramp the basic maneuver is a diverge, that is, a single traffic stream separating into two separate streams."* This study assumes that the on-ramp and off-ramp flows are uninterrupted by the up or down stream links. To approximate the ramp junction capacity, we first identify factors sensitive to the ramp junction capacity and then analyze the impact of those factors through a series of simulations. This chapter is organized into the following sections:

- Factors Sensitive to Capacities at Ramp Junctions
- Experimental Design
- Freeway Capacity with Single On-Ramp Junction
- Freeway Capacity with Single Off-Ramp Junction

## 3.1 Factors Sensitive to Capacity at Ramp Junction

Factors sensitive to the capacity of a freeway ramp junction are analyzed in this chapter. Similar to the analysis for the basic freeway section, factors sensitive to the ramp junction capacity are also categorized in the same way. In Type I, factors which can not be determined from the real world data are included, such as the car following sensitivity factor, the driver reaction time, and the hiatus period. In Type II, all factors that can be obtained from field survey are considered, (i.e., the grade percentage, the truck volumes, the auxiliary lane length, and the merging/diverging flows, etc.). In this chapter, we only focused on analyzing two main factors in Type II: (1) the ramp and mainline flows and (2) the length of auxiliary lane. Other factors belonging to Type I are still kept as their default values. The relations between the ramp junction capacity and the auxiliary lane length for various ramp flows are well established after analyzing results generated by CORSIM, which can greatly help traffic engineers to estimate capacities at ramp junctions capacity and to determine adequate (cost effective) lengths for auxiliary lanes while satisfying the desired service flow rates.

## 3.2 Experimental Design

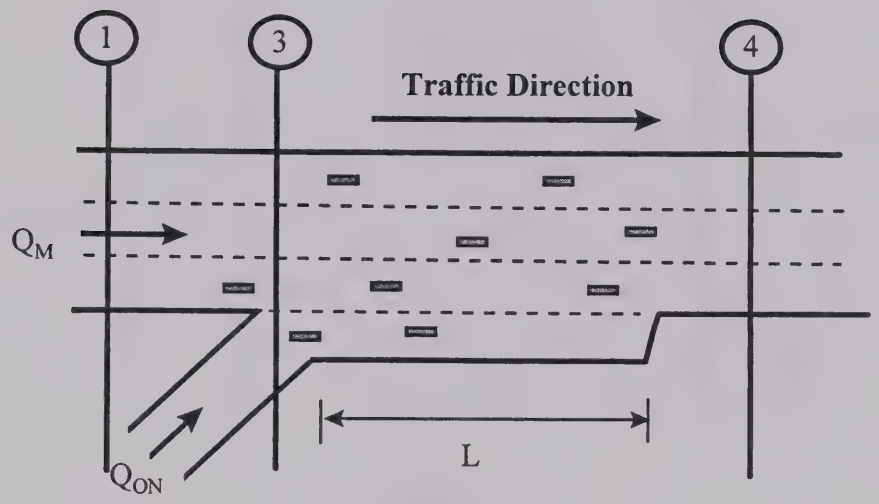
Freeways (two-lane, three-lane, and four-lane cases) with a single on-ramp or a single off-ramp are established and shown in Figures 3-1 and 3-2 . The capacities at the on-ramp and the off-ramp junctions are observed at nodes 4 and 3, respectively. Lengths

of links are kept the same throughout all analyzed networks, while the average mainline and ramp free-flow speeds are considered as 65 mph and 45 mph, respectively.

Before analyzing the on-ramp junction capacity, two scenarios of flow combinations (entry and merging/diverging flows) are considered. In Scenario 1, the sum of mainline and ramp flows, called the total flow, is designed as the capacity observed from the basic freeway section (i.e., the resulting total flows for 2, 3, and 4-lane cases are 4680, 7020 and 9360 pcph, respectively). Then, the ramp junction capacity is analyzed by altering the ramp and mainline flows complementarily. In Scenario 2, the analysis of the ramp junction capacity is based on fixed entry flow from mainline but changing the on-ramp flow.

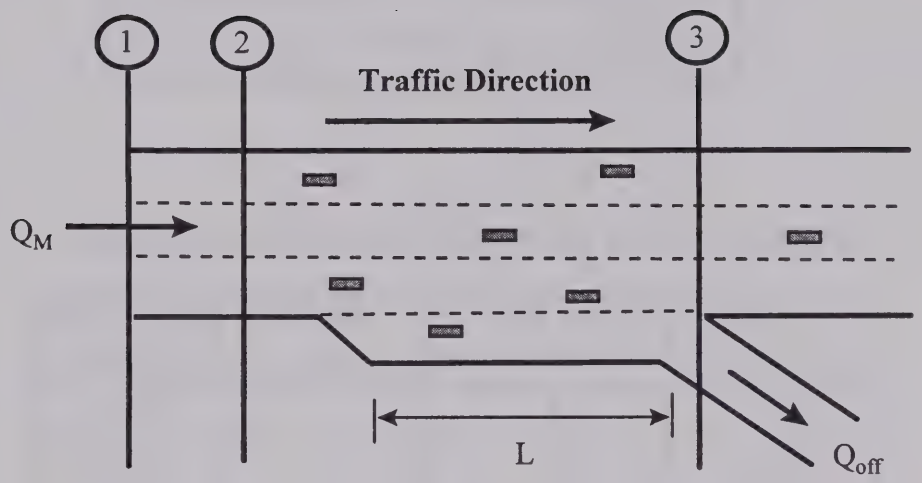
For off-ramp cases, fixed entry flows (i.e., the maximum discharged flow addressed in Chapter 2) and various off-ramp exiting flows (i.e., 4%, 8%, 12% and 16% of the entry flow) are analyzed. The auxiliary lane lengths (ALL) for both on-ramp and off-ramp junctions varies from 500 to 5000 feet with an increment of 500 feet for observing the ALL impact on the ramp junction capacity. From Tables 3-1 and 3-2, various combinations of on-ramp and off-ramp cases are considered and will be examined in sections 3.3 and 3.4.

The total number of simulations performed for analyzing on-ramp junction capacity is 720 (including 3 freeway configurations, 4 on-ramp flows, 6 auxiliary lane lengths and 10 one-hour observations), while there are the same efforts required for




$Q_M + Q_{ON} = 7020$ veh/h
$Q_{ON} = 250, 500, 750, 1000$ veh/h
L =500, 1000, 1500, 2000 ft

**Figure 3-1 Configuration of Single Lane On-Ramp Junction (3-Lane Case)**




$Q_M$ = 7020 veh.
$Q_{off}$ = 4, 8, 12, 16 % of $Q_M$
L = 500, 1000, 1500, 2000 ft

**Figure 3-2 Configuration of Single Lane Off-Ramp Junction (3-Lane Case)**

**Table 3-1 Variables Used in On-Ramp Capacity Analysis**

| Freeway Lanes | Ramp Flow | Auxiliary Lane Length (ft) |
|---|---|---|
| 2 | 250 | 500 |
| 3 | 500 | 1000 |
| 4 | 750 | 1500 |
| - | 1000 | 2000 |
| - | - | 2500 |
| - | - | 3000 |

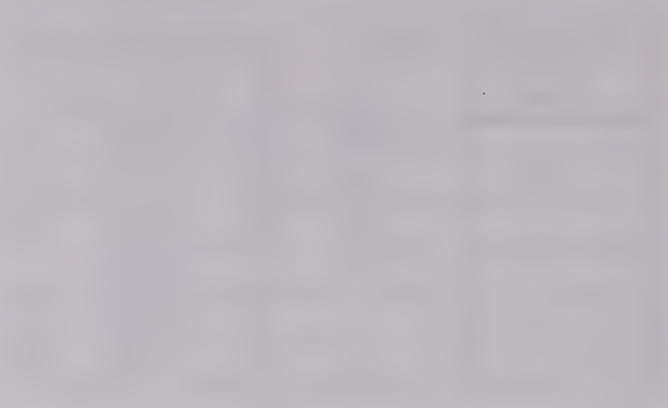
**Table 3-2 Variables Used in Off-Ramp Capacity Analysis**

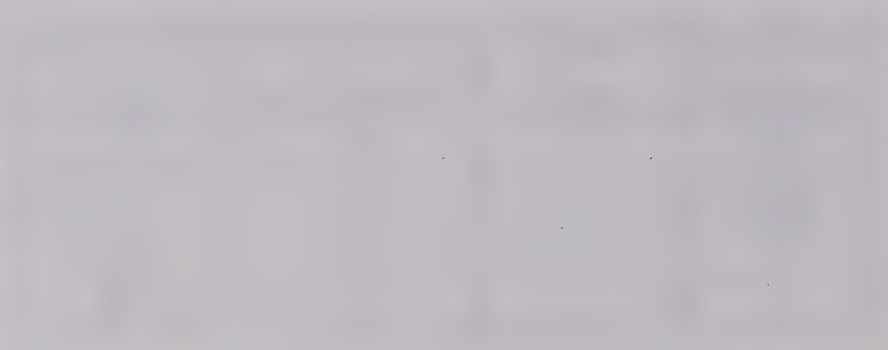
| Warning Sign Location (ft) | Freeway Lanes | Ramp Flow (% of Entry Flow) | Auxiliary Lane Length (ft) |
|---|---|---|---|
| 2500 | 2 | 4 | 500 |
| 4000 | 3 | 8 | 1000 |
| 6000 | 4 | 12 | 1500 |
| - | - | 16 | 2000 |
| - | - | - | 2500 |

analyzing off-ramp cases. Therefore, the total number of simulations required in this chapter is 1440.

## 3.3 Freeway Capacity with Single On-Ramp

For approximating the on-ramp junction capacity, simulation results obtained from CORSIM for Scenario 1 are summarized from Tables 3-3 through 3-5 and shown in





Figures 3-3 through 3-5. Conclusions and findings, after investigating the simulation results are made and discussed below.

1. The critical ALL for achieving the maximum flow passing through node 4 depends on the number of lanes on the mainline, which increases with the decreasing number of lanes. Since the number of lanes on the mainline increases, vehicles on the mainline have more opportunities to shift towards the left lane (The movement for vacating the right lane, called anticipatory lane changes) before reaching the on-ramp junction. Thus, gaps could be easier found as vehicles merging from the ramp. The critical ALLs are observed and approximately 3000, 1500, and 1000 feet for two-lane, three-lane and four-lane freeways, respectively. However, the 1994-HCM suggests that the adequate auxiliary lane length is 1500 feet, irrespective of the number of lanes on the mainline.

2. The magnitude of the on-ramp flow will affect the on-ramp junction capacity when the ALL is shorter than its critical length. For example, in two-lane cases as shown in Table 3-6, the critical ALL is 1500 feet. If the ALL is less than 1500 feet, the increasing on-amp flow causes capacity reduction at the junction.

3. If other factors sensitive to the ramp junction capacity are fixed and if the ALL is fixed and shorter than the critical length, the on-ramp junction capacity increases as the number of lanes on the mainline increases,. For example, when ALL is 500 feet as shown in Figures 3-3 through 3-5, the on-ramp junction capacities of two-lane, three-

lane and four-lane cases are 2153, 2290, and 2345 vphpl, respectively as, the on-ramp flow is fixed and 250 vph.

4. As the ALL is greater than the critical length, the influence of the on-ramp flow to the junction capacity becomes insignificant, as shown in Table 3-5 and Figure 3-5. However, the on-ramp junction capacities for four-lane cases are almost constant (approximately 2340 vph) when the ALL varies from 1000 to 2500 feet.

The generated simulation results, based on Scenario 2, are summarized in Tables 3-6 through 3-8 and shown in Figures 3-6 through 3-8. The on-ramp junction capacity increases as the ALL increases. It exceeds total entry flow (the maximum observed flow for basic freeway section). For instance, in Figure 3-6, observed hourly flow exceeds the capacity identified for basic freeway sections if the ALL is 3000 feet or more. Since the observed flow passing through node 4 does not converge while increasing the on-ramp flow, we could not find the critical ALL for all freeway ramp junctions. It is a very important issue for further investigation deficiencies and flows in CORSIM.

**Table 3-3 Capacity at On-Ramp Junctions (Scenario 1, 2-Lane Case)**

| | Ramp Flow (pcphpl) | | | |
|---|---|---|---|---|
| *ALL (ft) | 250 | 500 | 750 | 1000 |
| 500 | 2159 (7.41) | 2092 (6.10) | 2044 (8.14) | 2018 (6.81) |
| 1000 | 2242 (9.76) | 2195 (17.64) | 2168 (8.56) | 2146 (6.26) |
| 1500 | 2285 (5.5) | 2256 (10.85) | 2237 (8.31) | 2227 (12.77) |
| 2000 | 2315 (10.58) | 2300 (7.95) | 2283 (13.34) | 2271 (9.70) |
| 2500 | 2336 (5.06) | 2321 (7.9) | 2310 (6.21) | 2309 (6.43) |
| 3000 | 2339 (4.05) | 2339 (5.12) | 2336 (10.20) | 2337 (11.03) |

*ALL (ft) = Auxiliary Lane Length (ft)

**Table 3-4 Capacity at On-Ramp Junctions (Scenario 1, 3-Lane Case)**

| | Ramp Flow (pcphpl) | | | |
|---|---|---|---|---|
| ALL (ft) | 250 | 500 | 750 | 1000 |
| 500 | 2289 (4.41) | 2242 (3.77) | 2211 (5.18) | 2198 (6.67) |
| 1000 | 2333 (4.54) | 2312 (4.49) | 2289 (4.67) | 2276 (9.55) |
| 1500 | 2334 (2.45) | 2334 (4.68) | 2332 (6.70) | 2327 (8.59) |
| 2000 | 2335 (2.26) | 2335 (2.11) | 2334 (3.24) | 2333 (3.84) |
| 2500 | 2335 (2.67) | 2335 (1.59) | 2335 (3.0) | 2334 (2.77) |

**Table 3-5 Capacity at On-Ramp Junctions (Scenario 1, 4-Lane Case)**

| | Ramp Flow (pcphpl) | | | |
|---|---|---|---|---|
| ALL (ft) | 250 | 500 | 750 | 1000 |
| 500 | 2339 (2.93) | 2310 (6.55) | 2283 (4.59) | 2274 (7.12) |
| 1000 | 2339 (2.19) | 2339 (3.22) | 2337 (2.74) | 2335 (4.85) |
| 1500 | 2340 (1.66) | 2339 (3.68) | 2339 (4.21) | 2339 (4.21) |
| 2000 | 2340 (1.54) | 2340 (2.33) | 2339 (3.43) | 2338 (2.86) |
| 2500 | 2340 (1.42) | 2340 (1.86) | 2339 (2.16) | 2339 (4.74) |

**Table 3-6 Capacity at On-Ramp Junctions (Scenario 2, 2-Lane Case)**

| | Ramp Flow (pcphpl) | | | |
|---|---|---|---|---|
| ALL(ft) | 250 | 500 | 750 | 1000 |
| 500 | 2153 (7.45) | 2084 (5.62) | 2042 (6.19) | 2018 (8.81) |
| 1000 | 2241 (8.55) | 2195 (6.55) | 2165 (8.06) | 2144 (9.40) |
| 1500 | 2285 (10.00) | 2256 (5.50) | 2240 (7.31) | 2222 (8.34) |
| 2000 | 2318 (7.45) | 2299 (10.13) | 2278 (6.08) | 2272 (8.67) |
| 2500 | 2337 (11.43) | 2325 (7.43) | 2313 (6.82) | 2306 (6.96) |
| 3000 | 2357 (8.51) | 2344 (7.97) | 2335 (13.14) | 2330 (9.30) |
| 3500 | 2362 (7.07) | 2366 (4.39) | 2358 (8.02) | 2353 (9.14) |
| 4000 | 2378 (7.45) | 2380 (8.04) | 2374 (8.45) | 2373 (8.62) |
| 4500 | 2385 (10.89) | 2389 (7.67) | 2388 (4.37) | 2389 (5.39) |
| 5000 | 2394 (8.69) | 2403 (8.14) | 2398 (10.35) | 2399 (7.86) |

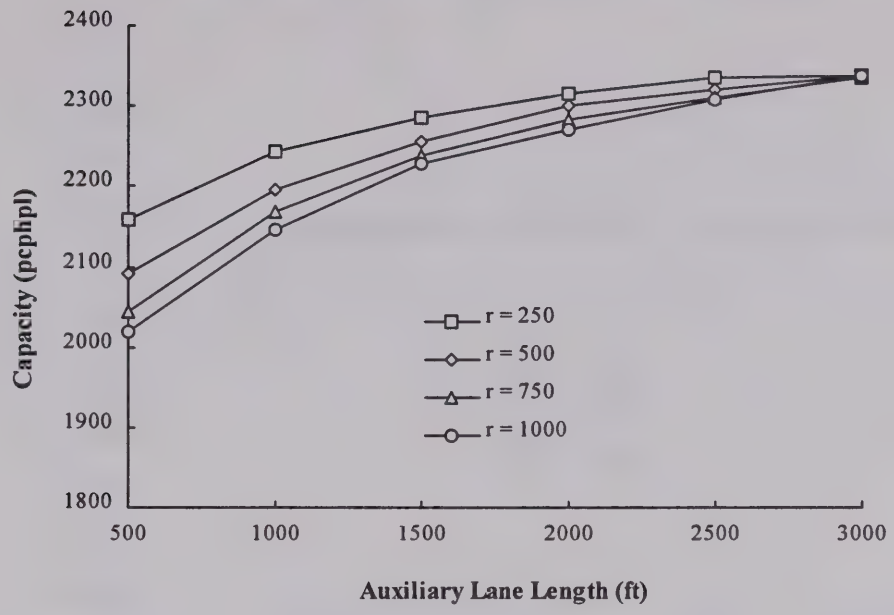
**Table 3-7 Capacity at On-Ramp Junctions (Scenario 2, 3-Lane Case)**

| | Ramp Flow (pcphpl) | | | |
|---|---|---|---|---|
| ALL (ft) | 250 | 500 | 750 | 1000 |
| 500 | 2290 (5.10) | 2241 (6.78) | 2210 (6.47) | 2194 (5.78) |
| 1000 | 2346 (5.28) | 2312 (7.26) | 2295 (6.63) | 2276 (8.14) |
| 1500 | 2372 (6.44) | 2350 (7.67) | 2339 (9.53) | 2333 (7.88) |
| 2000 | 2394 (6.18) | 2377 (5.89) | 2369 (6.64) | 2364 (7.10) |

**Table 3-8 Capacity at On-Ramp Junctions (Scenario 2, 4-Lane Case)**

| | Ramp Flow (pcphpl) | | | |
|---|---|---|---|---|
| ALL (ft) | 250 | 500 | 750 | 1000 |
| 500 | 2346 (2.95) | 2311 (4.56) | 2289 (6.92) | 2273 (7.10) |
| 1000 | 2385 (5.26) | 2367 (7.51) | 2346 (4.44) | 2340 (4.83) |
| 1500 | 2398 (4.38) | 2398 (4.86) | 2384 (6.10) | 2373 (7.90) |
| 2000 | 2398 (5.47) | 2412 (4.65) | 2404 (6.61) | 2402 (5.39) |
| 2500 | 2398 (4.40) | 2425 (6.08) | 2421 (5.29) | 2419 (5.34) |
| 3000 | 2398 (3.77) | 2434 (4.9) | 2436 (4.28) | 2430 (7.62) |
| 3500 | - | 2447 (5.92) | 2446 (7.10) | 2441 (6.35) |
| 4000 | - | 2453 (6.49) | 2452 (3.18) | 2446 (5.18) |
| 4500 | - | 2456 (7.81) | 2457 (6.17) | 2454 (5.16) |
| 5000 | - | 2459 (2.25) | 2467 (4.22) | 2464 (7.00) |

**Figure 3-3 Capacity at On-Ramp Junction vs. ALL for Various Ramp Flows (Scenario 1, 2-Lane Case)**




Note: r=Ramp flow (vph)

**Figure 3-4 Capacity at On-Ramp Junction vs. ALL for Various Ramp Flows ( Scenario 1, 3-Lane Case)**

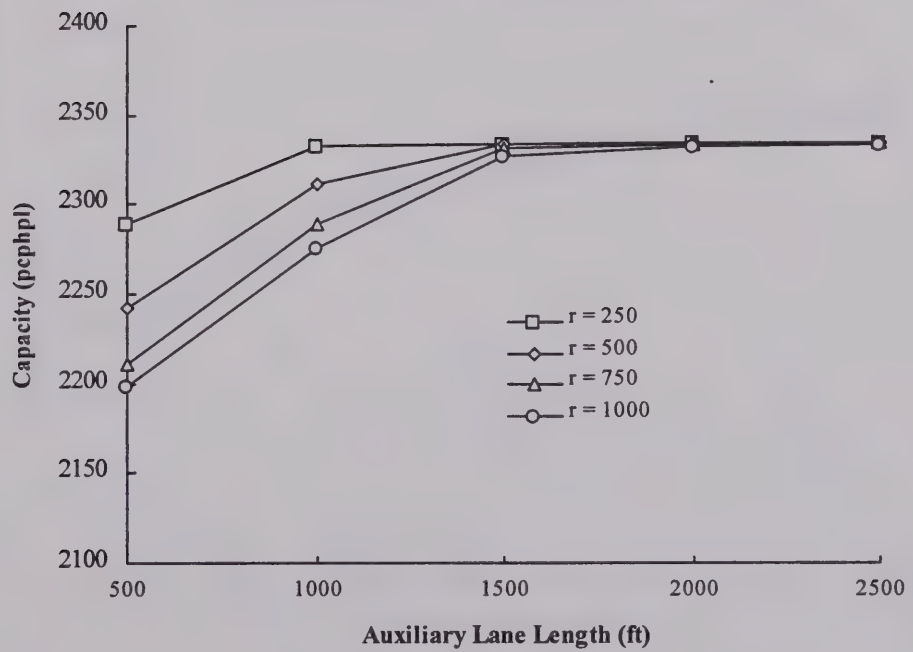


**Figure 3-5 Capacity at On-Ramp Junction vs. ALL for Various Ramp Flows ( Scenario 1, 4-Lane Case)**

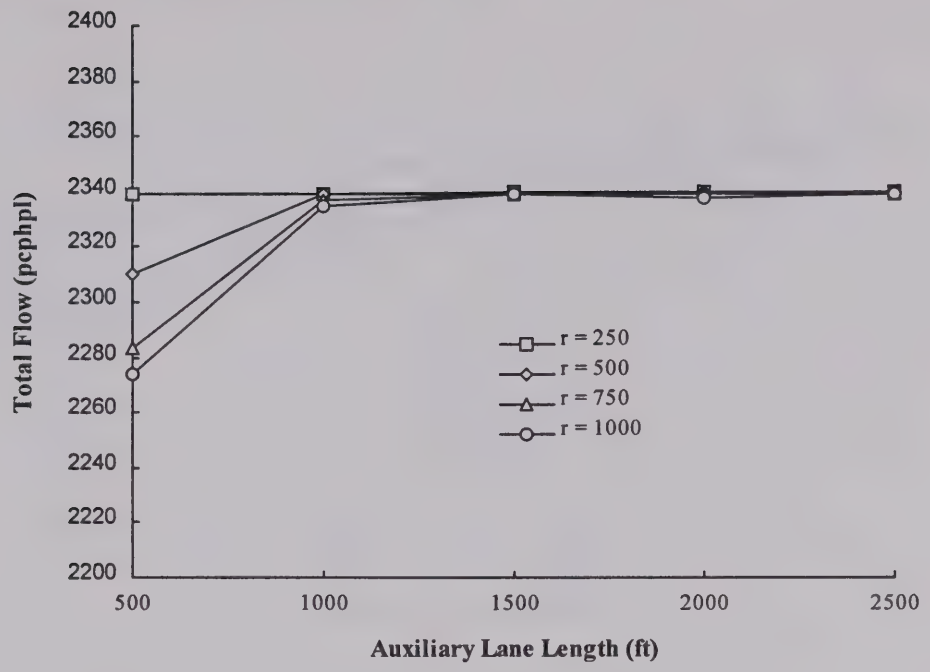



**Figure 3-6 Capacity at On-Ramp Junction vs. ALL for Various Ramp Flows ( scenario 2, 2-Lane Case)**

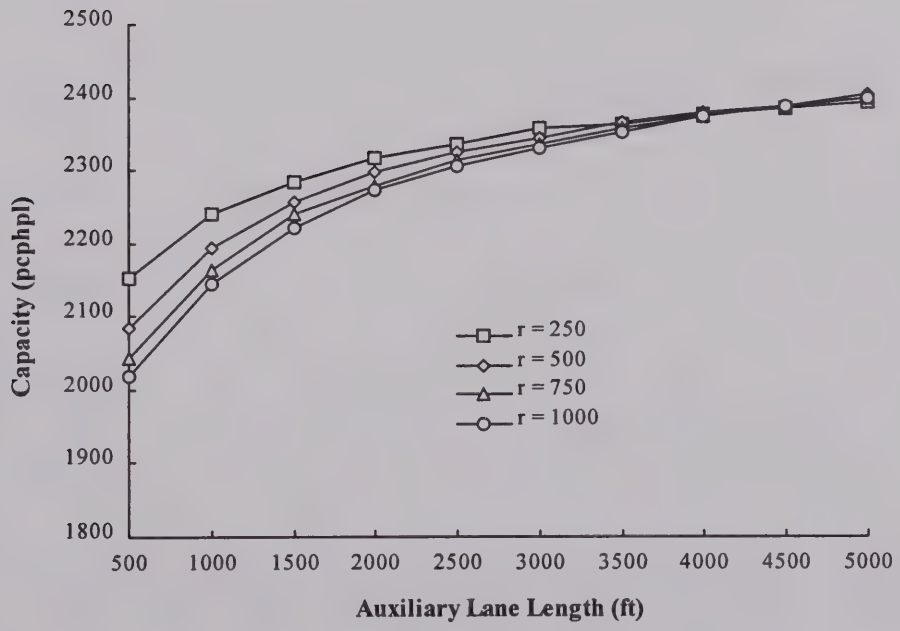


**Figure 3-7 Capacity at On-Ramp Junction vs. ALL for Various Ramp Flows (Scenario 2, 3-Lane Case)**

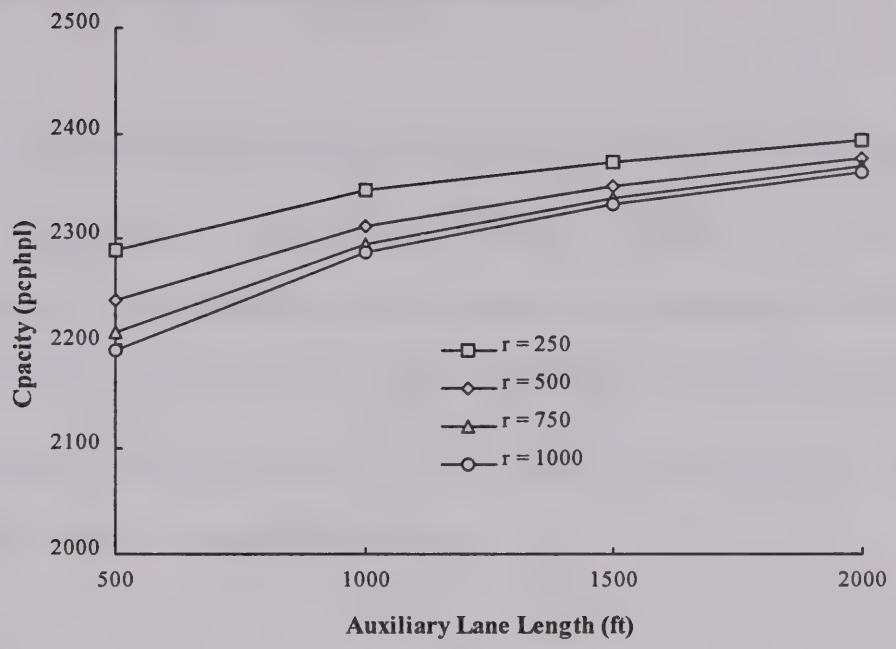
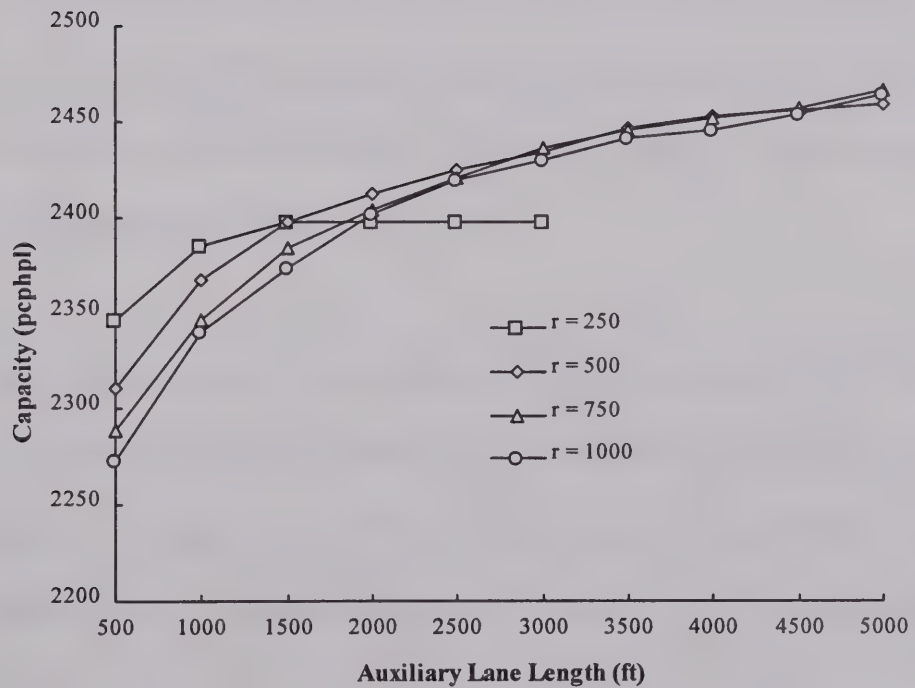



**Figure 3-8 Capacity at On-Ramp Junction vs. ALL for Various Ramp Flows(Scenario 2, 4-Lane Case)**



## 3.4 Freeway Capacity with Single Off-Ramp

The simulation results generated for various off-ramp junctions are summarized in Tables 3-9 through 3-17 and shown in Figures 3-9 through 3-17. Similar to on-ramp cases, capacities of off-ramp junctions are approximated considering various deceleration ALLs and off-ramp flows. The impact of the distance between the off-ramp and its warning sign on the junction capacity is analyzed by altering that distance from 2500 to 6000 feet. Results are concluded as follows:

1. The maximum flows at off-ramp junctions (two-lane cases) are observed as the ALL is beyond 1500 feet, while considering that the off-ramp flow increases from 4 to 16 % of the total entry flow. Tables from 3-9 through 3-11 and Figures from 3-9 through 3-11 show that the off-ramp junction capacity of a two-lane freeway is pretty close to the capacity of the basic freeway section. This findings is very consistent with the findings addressed in HCM.

2. The 500-foot auxiliary lane is sufficient enough to achieve the off-ramp junction capacity when the off-ramp flow is below 4 % (188 pcph) of the total entry flow (see Figures 3-9 through 3-17). If the number of lanes on the mainline increases, the critical ALL also increases (see Figures 3-9 through 3-17). This may be caused by the

increased number of lanes which increases the number of lane changes for exiting vehicles.

3. For higher off-ramp flows (i.e., 8, 12, and 16 % of the total entry flow), the critical ALLs for two-lane freeways range between 1000 and 1500 feet (see Figures 3-9 through 3-11). However, if the number of lanes on the mainline increases (for three and four-lane freeways) and the critical length exceeds 1500 feet, the capacity does increase but not significant. (see Figures 3-12 through 3-17).

4. There is very minor impact of the warning sign location on the off-ramp junction capacity for two-lane cases (see Figures 3-9 through 3-11). However, for three or four-lane freeways, the capacity increases significantly as the warning sign distance (WSD) increases (The WSD is defined as the distance between the off-ramp and the waning sign location). The optimal warning sign distance for which achieving the maximum off-ramp junction capacity should be studied in future. (see Figures 3-12 through 3-17).

5. The location of the off-ramp warning sign becomes sensitive to capacity when the number of lanes on the mainline increases. As the WSD increases, turbulence due to changing lanes incurred by the vehicles exiting to the off-ramp decreases. Again, as the number of lanes on the mainline increases, the number of lane changes incurred by exiting vehicles increases (see Figures 3-12 through 3-17).

**Table 3-9 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (2-Lane Case, WSD** = 2500 ft )**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL (ft)* | 4% | 8% | 12% | 16% |
| 500 | 2344 (4.32) | 2340 (9.50) | 2267 (23.79) | 2213 (20.97) |
| 1000 | 2343 (4.15) | 2346 (9.11) | 2347 12.67) | 2324 (16.47) |
| 1500 | 2342 (7.09) | 2346 6.31) | 2348 (15.29) | 2341 (17.99) |
| 2000 | 2344 (3.27) | 2348 (4.35) | 2344 (25.83) | 2349 (7.92) |
| 2500 | 2343 (5.79) | 2346 (6.56) | 2348 (4.23) | 2349 (7.05) |

*ALL means auxiliary lane length

**WSD means warning sign distance

**Table 3-10 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (2-Lane Case, WSD = 4000 ft )**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL(ft) | 4% | 8% | 12% | 16% |
| 500 | 2341 (4.48) | 2338 (8.71) | 2285 (21.31) | 2227 (30.90) |
| 1000 | 2343 (8.64) | 2346 (2.90) | 2349 (11.12) | 2340 (14.32) |
| 1500 | 2343 (3.84) | 2345 (5.83) | 2348 (5.86) | 2346 (8.02) |
| 2000 | 2342 (5.84) | 2345 (6.00) | 2348 (3.65) | 2348 (8.32) |
| 2500 | 2344 (3.31) | 2345 (4.60) | 2350 (2350) | 2347 (5.38) |

**Table 3-11 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (2-Lane Case. WSD = 6000 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL(ft) | 4% | 8% | 12% | 16% |
| 500 | 2341 (4.18) | 2343 (6.17) | 2300 (18.04) | 2243 (10.90) |
| 1000 | 2342 (2.86) | 2345 (3.75) | 2347 (4.07) | 2347 (9.41) |
| 1500 | 2342 (3.04) | 2346 (7.38) | 2346 (5.68) | 2346 (7.10) |
| 2000 | 2342 (2.34) | 2346 (2.78) | 2345 (4.40) | 2348 (7.30) |
| 2500 | 2341 (4.67) | 2346 (7.05) | 2348 (2.24) | 2348 (5.99) |

**Table 3-12 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (3-Lane Case, WSD = 2500 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL(ft) | 4% | 8% | 12% | 16% |
| 500 | 2297 (22.73) | 2167 (11.47) | 2100 (9.30) | 2066 (13.60) |
| 1000 | 2299 (16.63) | 2213 (18.49) | 2157 (11.62) | 2137 (10.10) |
| 1500 | 2315 (15.16) | 2223 (13.69) | 2176 (12.69) | 2164 (12.75) |
| 2000 | 2318 (17.44) | 2227 (21.70) | 2188 (11.75) | 2185 (15.40) |
| 2500 | 2325 (15.31) | 2217 (16.76) | 2203 (10.73) | 2201 (13.90) |
| 3000 | 2319 (18.91) | 2235 (18.40) | 2219 (9.15) | 2225 (13.89) |

**Table 3-13 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (3-Lane Case, WSD = 4000 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL(ft) | 4% | 8% | 12% | 16% |
| 500 | 2304 (19.69) | 2195 (16.30) | 2147 (15.78) | 2112 (8.61) |
| 1000 | 2321 (19.93) | 2234 (19.16) | 2197 (7.58) | 2177 (11.69) |
| 1500 | 2328 (22.77) | 2251 (12.10) | 2214 (9.56) | 2212 (12.03) |
| 2000 | 2337 (19.89) | 2251 (22.87) | 2219 (12.95) | 2224 (5.51) |
| 2500 | 2329 (13.59) | 2252 (12.62) | 2235 (11.30) | 2236 (8.33) |
| 3000 | 2330 (16.98) | 2259 (11.23) | 2238 (12.26) | 2264 (19.86) |

**Table 3-14 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (3-Lane Case, WSD = 6000 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL(ft) | 4% | 8% | 12% | 16% |
| 500 | 2321 (19.18) | 2216 (9.10) | 2170 (13.64) | 2154 (14.63) |
| 1000 | 2323 (15.50) | 2248 (9.09) | 2223 (6.95) | 2210 (7.88) |
| 1500 | 2332 (19.03) | 2259 (15.70) | 2239 (17.24) | 2242 (10.58) |
| 2000 | 2338 (15.71) | 2270 (15.76) | 2250 (12.63) | 2253 (11.68) |
| 2500 | 2333 (10.54) | 2264 (17.92) | 2269 (17.24) | 2271 (12.59) |
| 3000 | 2333 (10.54) | 2268 (18.36) | 2263 (13.08) | 2283 (12.05) |

**Table 3-15 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (4-Lane Case, WSD = 2500 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL (ft) | 4% | 8% | 12% | 16% |
| 500 | 2215 (19.24) | 2089 (9.96) | 2029 (14.46) | 1978 (9.99) |
| 1000 | 2222 (15.32) | 2126 (11.96) | 2077 (10.99) | 2053 (14.46) |
| 1500 | 2229 (28.65) | 2142 (15.30) | 2096 (11.56) | 2076 (6.81) |
| 2000 | 2234 (22.03) | 2139 (14.71) | 2110 (14.41) | 2096 (8.97) |
| 2500 | 2235 (9.99) | 2145 (10.10) | 2118 (14.66) | 2114 (9.25) |

**Table 3-16 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (4-Lane Case, WSD = 4000 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL (ft) | 4% | 8% | 12% | 16% |
| 500 | 2226 (21.21) | 2128 (11.33) | 2073 (13.77) | 2038 (11.23) |
| 1000 | 2246 (12.83) | 2163 (12.29) | 2122 (8.83) | 2101 (8.51) |
| 1500 | 2242 (23.83) | 2173 (8.37) | 2144 (9.91) | 2127 (9.55) |
| 2000 | 2250 (17.45) | 2171 (16.63) | 2161 (12.44) | 2147 (7.69) |
| 2500 | 2256 (17.56) | 2186 (7.63) | 2157 (16.98) | 2165 (7.51) |

**Table 3-17 Capacity at Off-Ramp Junctions for Various ALLs and Exit Flows (4-Lane Case, WSD = 6000 ft)**

| | Percentages of Exit Flows | | | |
|---|---|---|---|---|
| ALL (ft) | 4% | 8% | 12% | 16% |
| 500 | 2247 (21.00) | 2153 (15.92) | 2106 (13.68) | 2080 (7.67) |
| 1000 | 2263 (18.65) | 2188 (19.99) | 2152 (12.08) | 2142 (6.54) |
| 1500 | 2269 (17.96) | 2205 (11.86) | 2172 (8.06) | 2176 (9.84) |
| 2000 | 2271 (20.93) | 2195 (18.87) | 2185 (8.69) | 2192 (10.47) |
| 2500 | 2277 (15.41) | 2205 (11.48) | 2196 (9.35) | 2206 (10.90) |

**Figure 3-9 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (2-Lane Case, WSD = 2500 ft )**

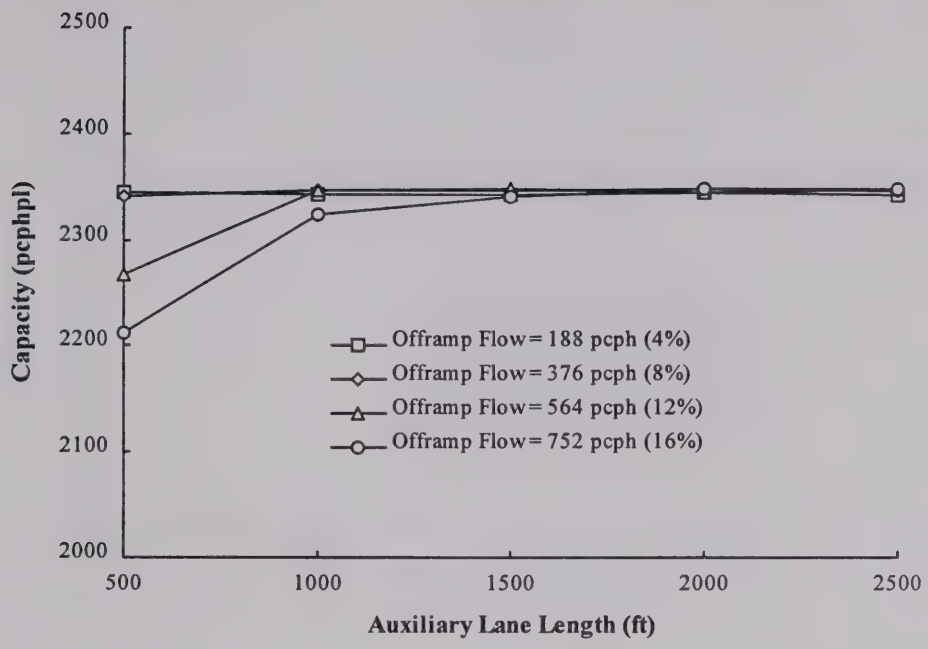



**Figure 3-10 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (2-Lane Case, WSD = 4000 ft)**

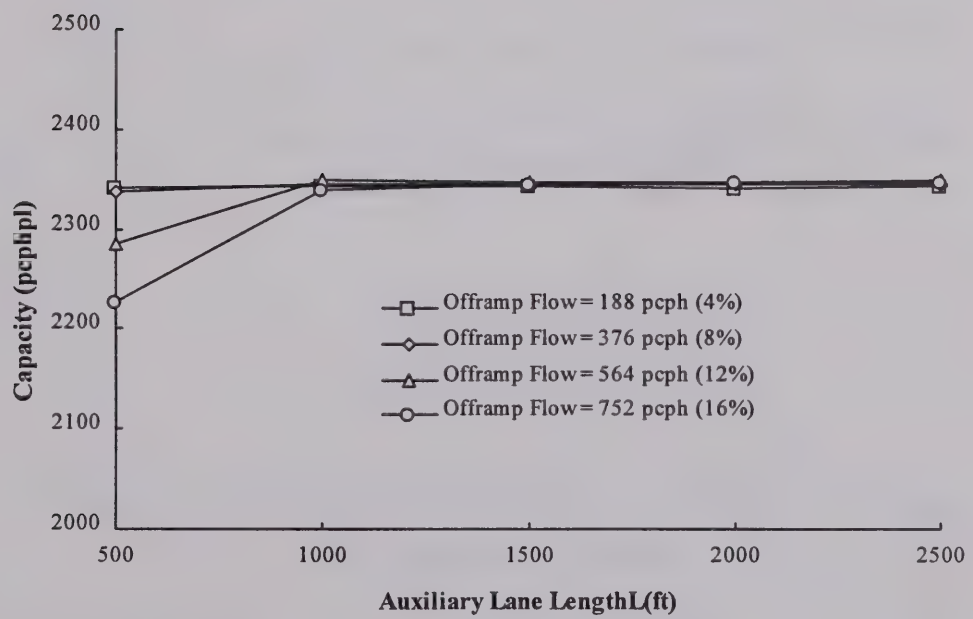


**Figure 3-11 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (2-Lane Case, WSD = 6000 ft)**

**Figure 3-12 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (3-Lane Case, WSD = 2500 ft to )**

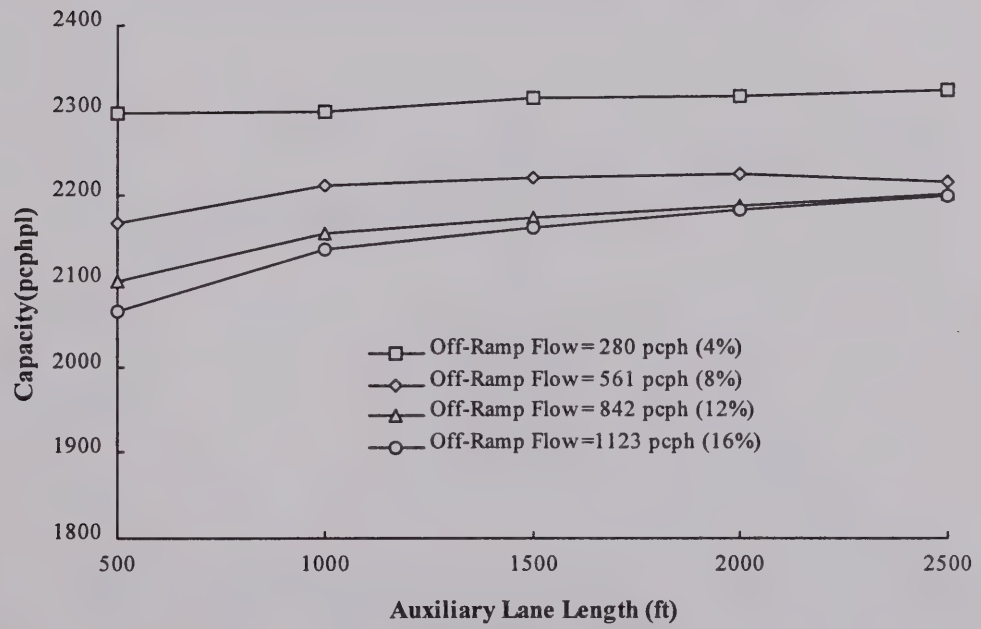


**Figure 3-13 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (3-Lane Case, WSD= 4000 ft)**

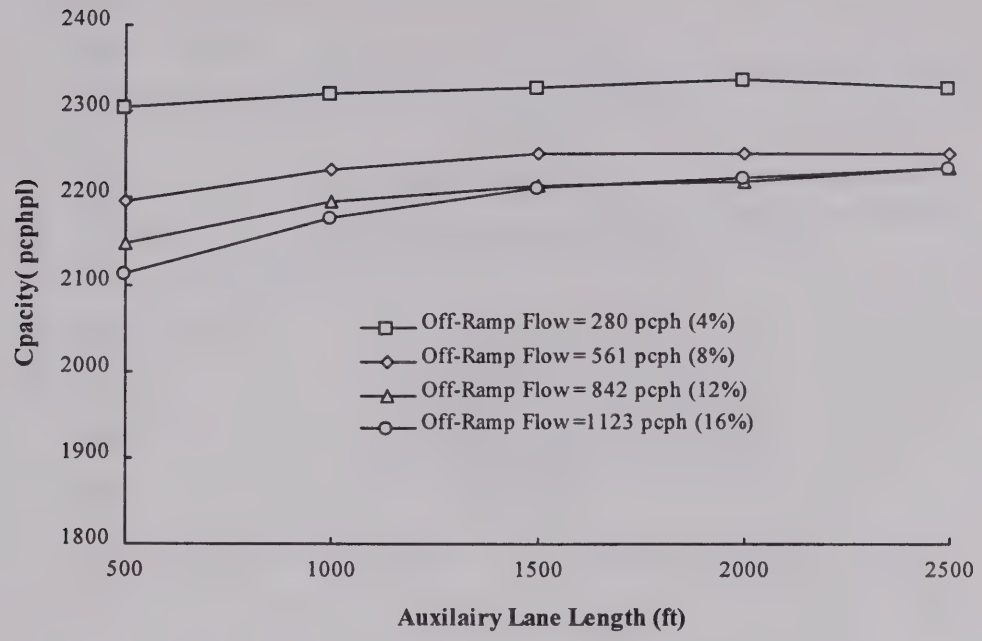



**Figure 3-14 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (3-Lane Case, WSD = 6000 ft)**

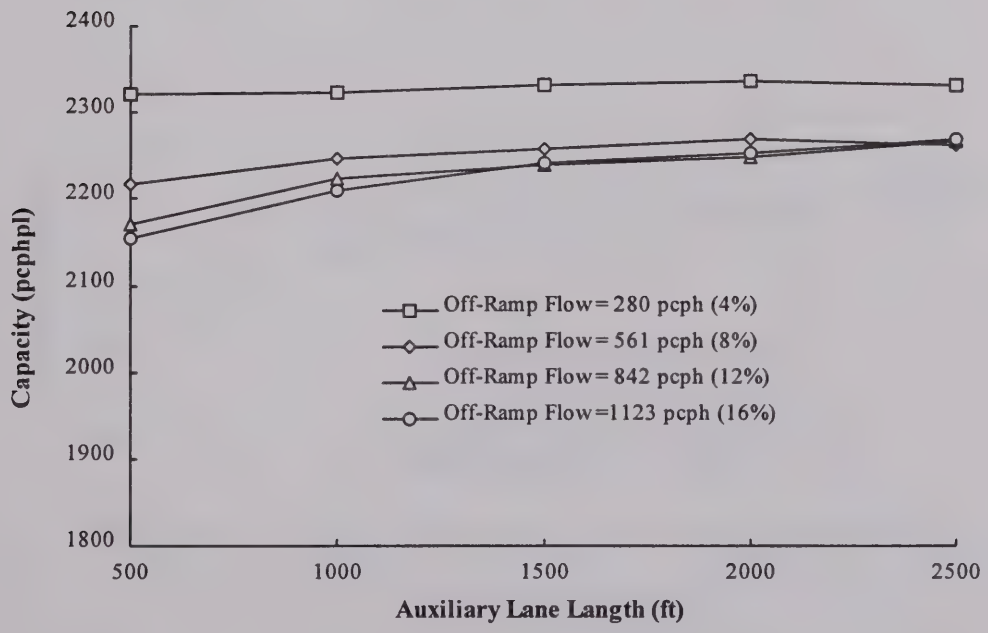


**Figure 3-15 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (4-Lane Case, WSD = 2500 ft)**

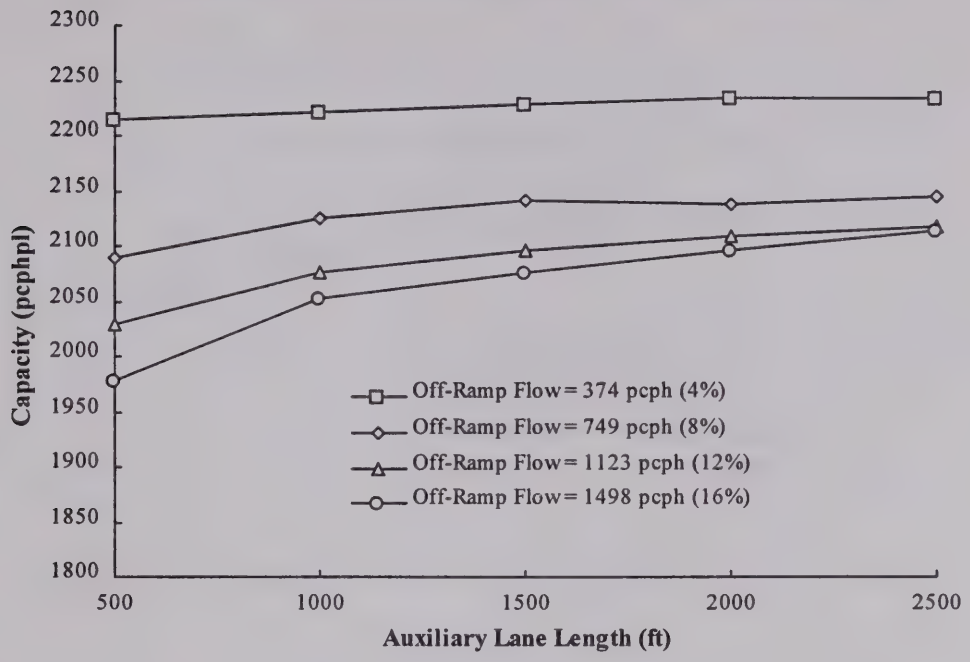



**Figure 3-16 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (4-Lane Case, WSD = 4000 ft)**

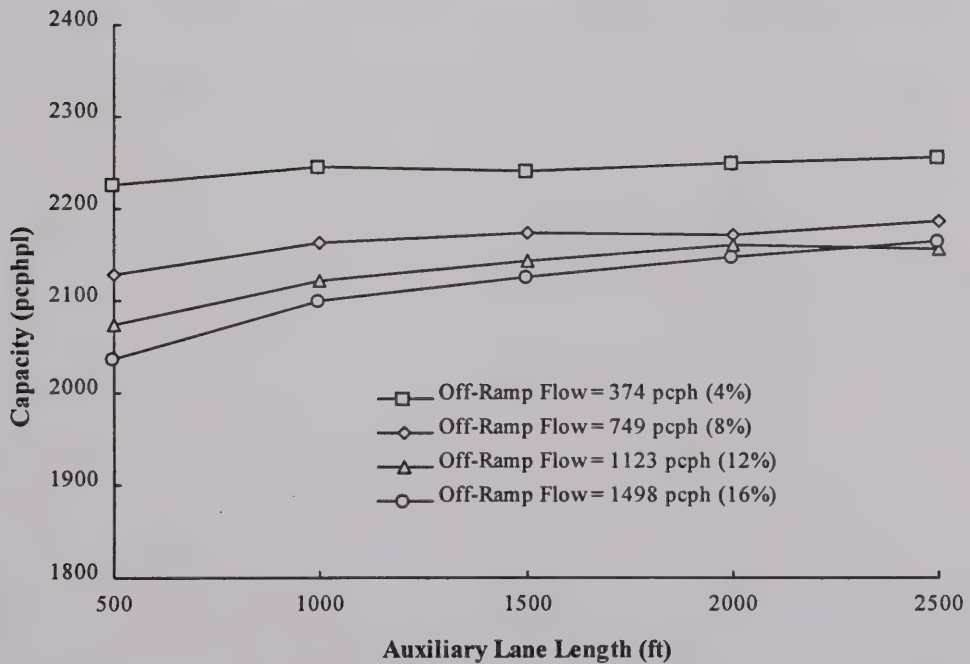
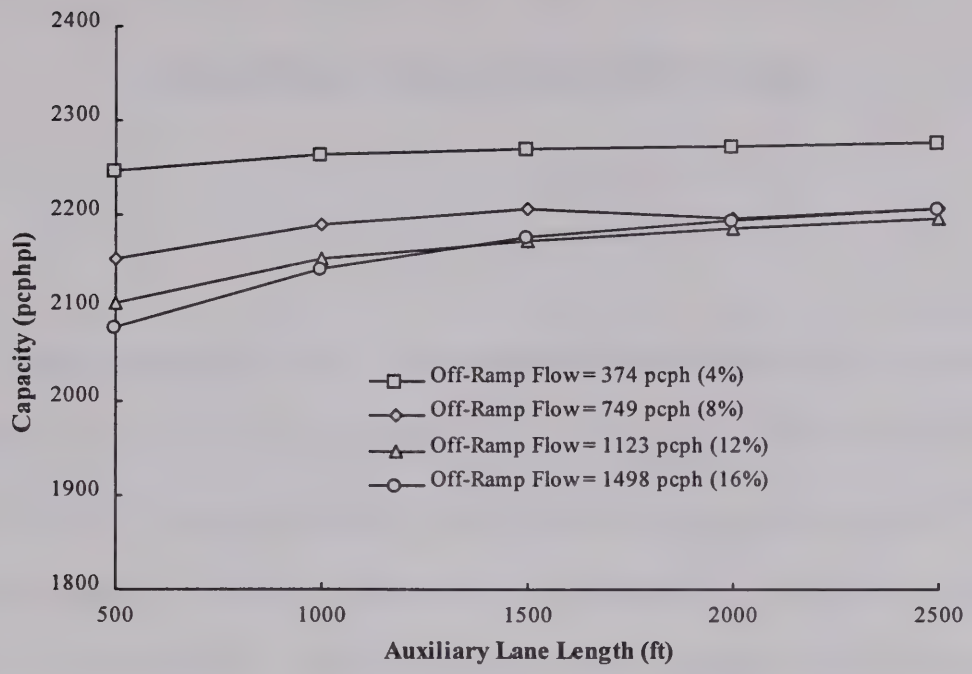


**Figure 3-17 Capacity at Off-Ramp Junction vs. ALL for Various Off-Ramp Flows (4-Lane Case, WSD = 6000 ft)**



# CHAPTER 4

## FREEWAY WEAVING SECTIONS

Weaving defined in HCM is *"the crossing of two or more traffic traveling in the same general direction along a significant length of highway, without the aid of traffic control devices."* Weaving sections are formed when the merging area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two joined by an auxiliary lane. In HCM, weaving areas are categorized into three different types depending on the minimum number of lane changes which must be made by weaving vehicles as they travel through the section. This chapter analyzes the capacity for the Type A weave area (see Figure 4-1) and is organized into the following sections.

- Factors Sensitive to Capacity in Weaving Areas.
- Experimental Design
- Capacity in Weaving Areas
- Speeds in Weaving Areas

## 4.1 Factors Sensitive to Capacity in Weaving Areas

In the development of guidelines for approximating the capacity of the Type A weaving area, factors sensitive to capacity (identified in Phase I) are selected for further analysis. Similar to basic freeway sections and ramp junctions, factors belonging to Type I include the car-following sensitivity factor, the driver reaction time, and the hiatus period. These factors need to be well calibrated before approximating the capacity by using simulation models. The Type II factors consist of the grade and truck percentages, the lane width, the number of lanes on the mainline, the weaving flow, the warning sign location, and the auxiliary lane length.

Terms used in this analysis include entry flow, through flow, weaving flow, weaving capacity, critical ALL and weaving section capacity and are defined as follows:

- Entry Flow: the hourly flow rate entering from node 8001 (See Figure 4-1)
- Through Flow: the hourly flow rate entering from node 8001 and exiting at node 8005
- Weaving Flow: the sum of ramp-to-freeway and freeway-to-ramp flows
- Weaving Capacity: the maximum observed weaving flow
- Critical ALL: the length of auxiliary lane; beyond which, the weaving flow has no or insignificant impact on the weaving section capacity.
- Weaving Section Capacity: the maximum observed flow passing through node 4 (including the flow exiting to the off-ramp)

While performing a pilot test before simulating weaving operation, abnormal situation when running CORSIM were found. Two vehicles (one merging from the on-ramp to the mainline and the other diverging from the mainline to the off-ramp) blocked the right lane on the freeway and the auxiliary lane in front of the off-ramp gore because of competing the right-of-way. This situation still exists even we specified 100% cooperative drivers who are willing to yield their right-of-way. Such a problem could generate biased results and restrained us to use CORSIM for further analysis. The updated CORSIM model was timely enhanced after reporting this problem to FHWA. After repeating the previous testing procedures, we found that the problems identified before were successfully fixed. Therefore, the CORSIM model used in this chapter is different from that used in Chapters 2 and 3.

## 4.2 Experimental Design

To demonstrate the relationship and the impact of factors on the capacity of weaving areas, an isolated weaving section is designed for the analysis and shown in Figure 4-1. The designed weaving section consists of the three lanes on mainline with a full auxiliary lane fed by a single lane on-ramp and feeding a single lane off-ramp. To evaluate the performance of traffic operations in weaving areas, four major factors influencing the capacity at weaving areas are identified, including the through flow, the weaving flow, and the auxiliary lane length (ALL).

Two scenarios were conducted for approximating capacity, including (1) increasing the on-ramp flow with fixed off-ramp demand and (2) increasing the off-ramp flow with fixed on-ramp demand. In both scenarios, the warning sign location is fixed and located at 1500 feet before the on-ramp gore, while the free-flow speeds of the mainline and the ramp are kept as 65 mph and 45 mph, respectively.

In Scenario 1, the designed entry flow from node 8001 varies from 5000 pcph to 7000 pcph including the off-ramp flow ranging between 400 pcph and 1260 pcph. In the mean time, the fixed on-ramp flow is 600 pcph. In Scenario 2, the flow entering from node 8001 is 7000 pcph including 840 pcph (12 % of 7000 pcph) exiting to the off-ramp, while the on-ramp flow varies from 300 pcph to 1000 pcph. Therefore, the resulting weaving flow ranges between 1140 and 1840 vph. For further considering comprehensively the impact of the ALL against the weaving capacity, the results for ALL varying from 1000 feet to 4000 feet in both scenarios are observed. Additionally, the weaving section capacity under various flow conditions and at different ALLs are monitored.

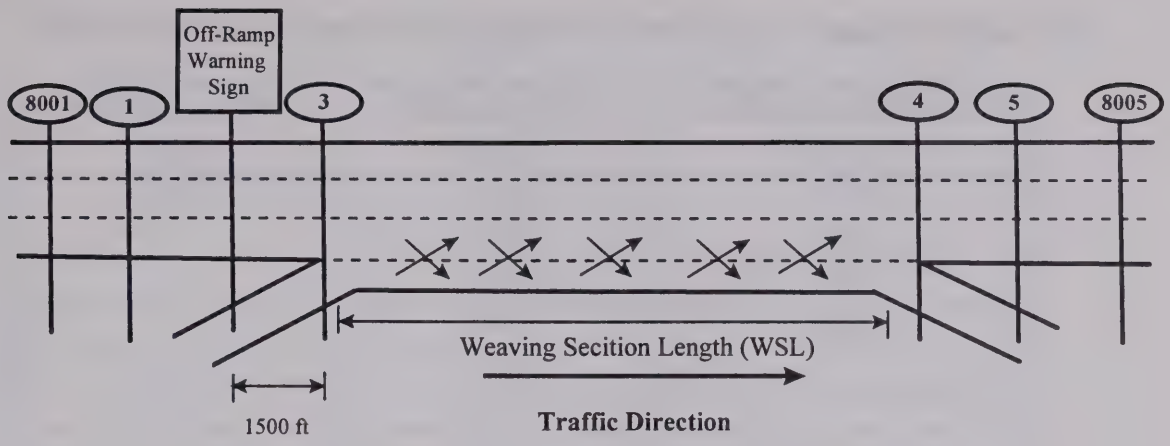



**Figure 4-1 Geometry of "Type A" Weaving Area**

**Table 4-1 Variables Used in Analyzing Weaving Section Capacity (Scenario 1)**

| Main Line Flow (pcph) | Diverging Flow (pcph) | Auxiliary Lane Length (ALL) (ft) |
|---|---|---|
| 5000 | 400 ((8%) | 1000 |
| | 800 (16%) | 2000 |
| | 1200 (24%) | 3000 |
| | | 4000 |
| 5714 | 400 (7%) | .1000 |
| | 800 (14%) | 2000 |
| | 1200 (21%) | 3000 |
| | | 4000 |
| 6667 | 400 (6%) | 1000 |
| | 800 (12%) | 2000 |
| | 1200 (18%) | 3000 |
| | | 4000 |
| 7000 | 420 (6%) | 1000 |
| | 840 (12%) | 2000 |
| | 1260 (18%) | 3000 |
| | | 4000 |

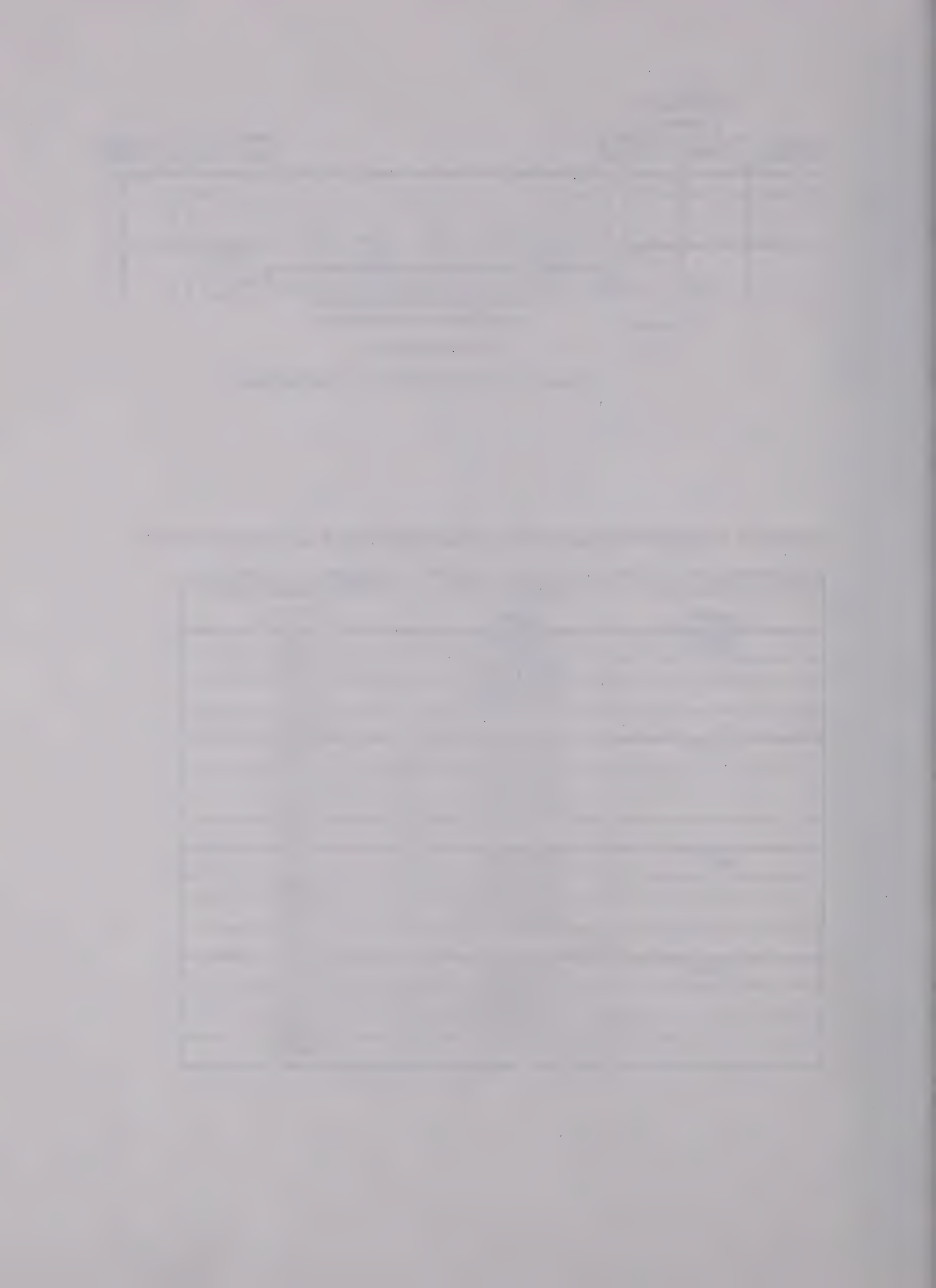


**Table 4-2 Variables Used in Analyzing Weaving Section Capacity (Scenario 2)**

| Main Line Flow (pcph) | Merging Flow (pcph) | Auxiliary Lane Length (ALL) (ft) |
|---|---|---|
| 7000 | 300 | 1000 |
| | 400 | 2000 |
| | 500 | 3000 |
| | 600 | 4000 |
| | 700 | |
| | 800 | |
| | 900 | |
| | 1000 | |

Factors and the associated values used in this analysis are summarized in Tables 4-1 and 4-2. The total number of simulation runs yields 800 including 480 runs for Scenario 1 (4 mainline flows, 3 diverging flows, 4 auxiliary lane lengths, and 10 observations for each combination) and 320 runs (1 mainline flow, 8 merging flows, 4 auxiliary lane lengths, and 10 observations for each combination ) for Scenario 2.

## 4.3 Capacity in Weaving Areas

Simulation results generated by CORSIM for Scenario 1 are summarized in Tables 4-3 through 4-5. The relations of the weaving section capacity and the weaving capacity against the ALL are shown in Figures 4-2 and 4-3. In Figure 4-2, the weaving section capacity increases while the ALL increases as the entry flows are high (e.g., 6667 vph or 7000 vph). However, as the entry flows are lower (e.g., 5000 vph or 5700 vph),

**Table 4-3 Weaving Section Capacity vs. ALL for Various Flow Conditions (Scenario 1)**

| *ALL (ft) | **5000 ***(400) | 5000 (800) | 5000 (1200) | 5700 (400) | 5700 (800) | 5700 (1200) | 6667 (400) | 6667 (800) | 6667 (1200) | 7000 (420) | 7000 (840) | 7000 (1260) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1000 | 5570 | 5580 | 5588 | 6297 | 6297 | 6306 | 6909 | 6905 | 6903 | 6897 | 6907 | 6898 |
| 2000 | 5567 | 5578 | 5586 | 6290 | 6298 | 6303 | 7049 | 7026 | 7046 | 7069 | 7029 | 7042 |
| 3000 | 5565 | 5580 | 5585 | 6290 | 6295 | 6302 | 7102 | 7076 | 7142 | 7143 | 7065 | 7127 |
| 4000 | 5566 | 5579 | 5585 | 6290 | 6297 | 6301 | 7144 | 7128 | 7194 | 7141 | 7124 | 7208 |

*ALL = Auxiliary Lane Length

** entry flow

*** diverging flow



**Table 4-4 Through Flow vs. ALL for Various Flow Conditions (Scenario 1)**

| *ALL (ft) | **5000 ***(400) | 5000 (800) | 5000 (1200) | 5700 (400) | 5700 (800) | 5700 (1200) | 6667 (400) | 6667 (800) | 6667 (1200) | 7000 (420) | 7000 (840) | 7000 (1260) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1000 | 4665 | 4329 | 3950 | 5387 | 5040 | 4673 | 5994 | 5653 | 5322 | 5985 | 5658 | 5330 |
| 2000 | 4627 | 4253 | 3881 | 5348 | 4962 | 4599 | 6100 | 5693 | 5341 | 6119 | 5709 | 5345 |
| 3000 | 4612 | 4216 | 3812 | 5338 | 4939 | 4546 | 6136 | 5712 | 5398 | 6162 | 5707 | 5373 |
| 4000 | 4607 | 4190 | 3783 | 5269 | 4914 | 4537 | 6158 | 5755 | 5425 | 6161 | 5745 | 5414 |

*ALL = Auxiliary Lane Length

**entry flow

*** diverging flow

**Table 4-5 Weaving Capacity vs. ALL for various Flow Conditions**

| *ALL (ft) | **5000 ***(400) | 5000 (800) | 5000 (1200) | 5700 (400) | 5700 (800) | 5700 (1200) | 6667 (400) | 6667 (800) | 6667 (1200) | 7000 (420) | 7000 (840) | 7000 (1260) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **1000** | 905 | 1251 | 1638 | 910 | 1257 | 1633 | 915 | 1252 | 1581 | 912 | 1249 | 1568 |
| **2000** | 940 | 1325 | 1705 | 942 | 1336 | 1704 | 949 | 1333 | 1705 | 950 | 1320 | 1697 |
| **3000** | 953 | 1364 | 1773 | 952 | 1356 | 1756 | 966 | 1364 | 1744 | 981 | 1358 | 1754 |
| **4000** | 959 | 1389 | 1802 | 1021 | 1383 | 1764 | 986 | 1373 | 1769 | 980 | 1379 | 1794 |

*ALL = Auxiliary Lane Length

**input flow

*** diverging flow

**Figure 4-2 Weaving Section Capacity vs. ALL for Various Flow Conditions (Scenario 1)**

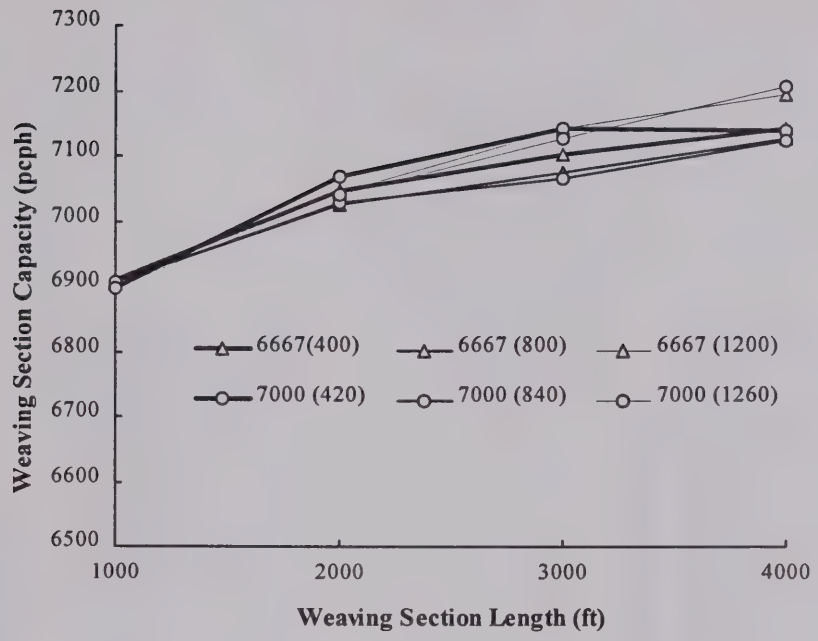



**Figure 4-3 Weaving Capacity vs. ALL for Various Flow Conditions (Scenario 1)**

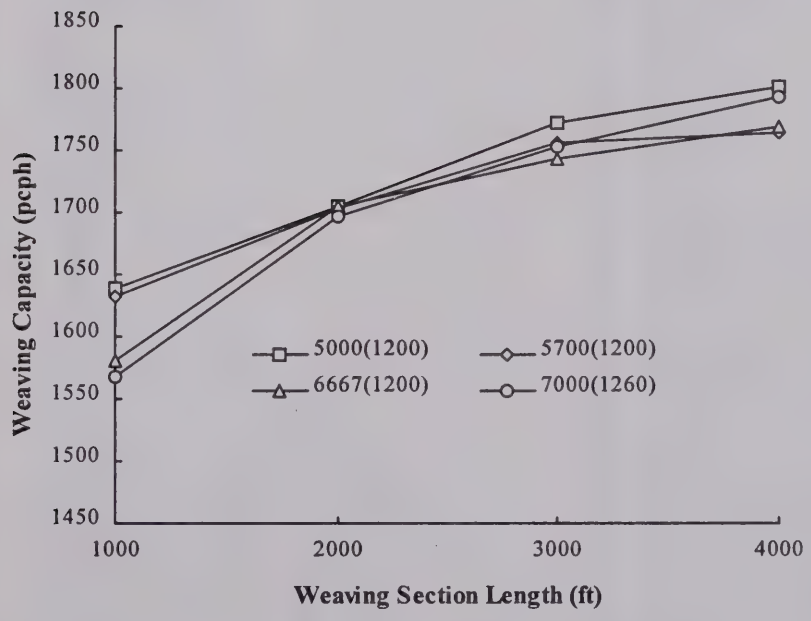


**Table 4-6 Weaving Capacity vs. ALL for Various Flow Conditions (Scenario 2)**

| **ALL (ft) | *7000(300) | 7000(400) | 7000(500) | 7000(600) | 7000(700) | 7000(800) | 7000(900) | 7000(1000) |
|---|---|---|---|---|---|---|---|---|
| 1000 | 891 | 994 | 1118 | 1196 | 1321 | 1421 | 1525 | 1621 |
| 2000 | 961 | 1086 | 1192 | 1288 | 1394 | 1504 | 1598 | 1708 |
| 3000 | 1000 | 1119 | 1217 | 1319 | 1413 | 1526 | 1624 | 1735 |
| 4000 | 1024 | 1131 | 1235 | 1337 | 1445 | 1556 | 1647 | 1756 |

*Auxiliary Lane Length (Merging Flow)
**Auxiliary lane length



**Table 4-7 Weaving Section Capacity vs. ALL for Various Flow Condition (Scenario 2)**

| **ALL (ft) | *7000(300) | 7000(400) | 7000(500) | 7000(600) | 7000(700) | 7000(800) | 7000(900) | 7000(1000) |
|---|---|---|---|---|---|---|---|---|
| 1000 | 6835 | 6872 | 6908 | 6920 | 6929 | 6947 | 6997 | 7023 |
| 2000 | 6912 | 6971 | 7014 | 7058 | 7103 | 7106 | 7152 | 7171 |
| 3000 | 7027 | 7024 | 7076 | 7078 | 7118 | 7173 | 7191 | 7225 |
| 4000 | 7058 | 7075 | 7055 | 7119 | 7171 | 7179 | 7230 | 7250 |

**Figure 4-4 Weaving Section Capacity vs. ALL for Various Flow Conditions (Scenario 2)**

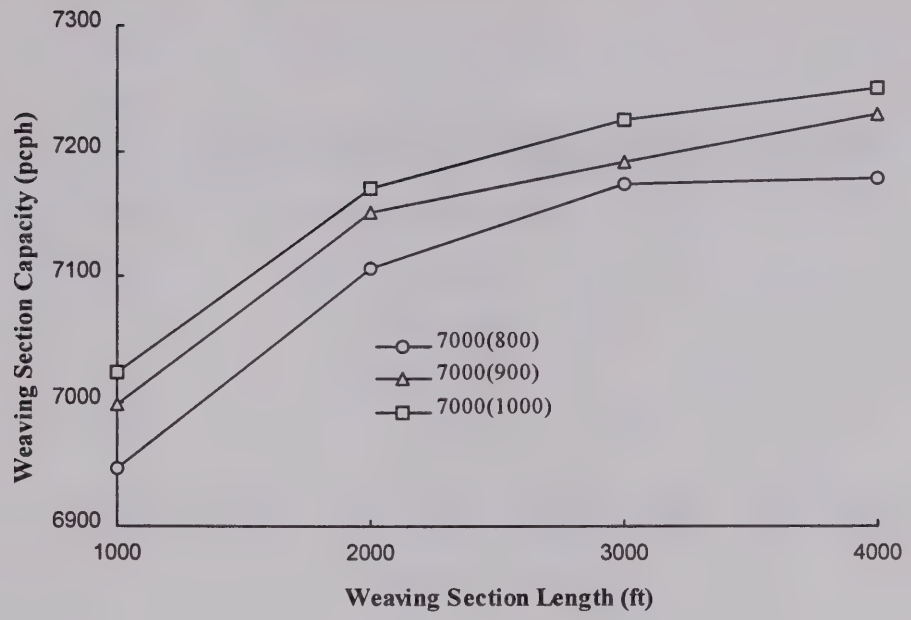



**Figure 4-5 Weaving Capacity vs. ALL for Various Flow Conditions (Scenario 2)**

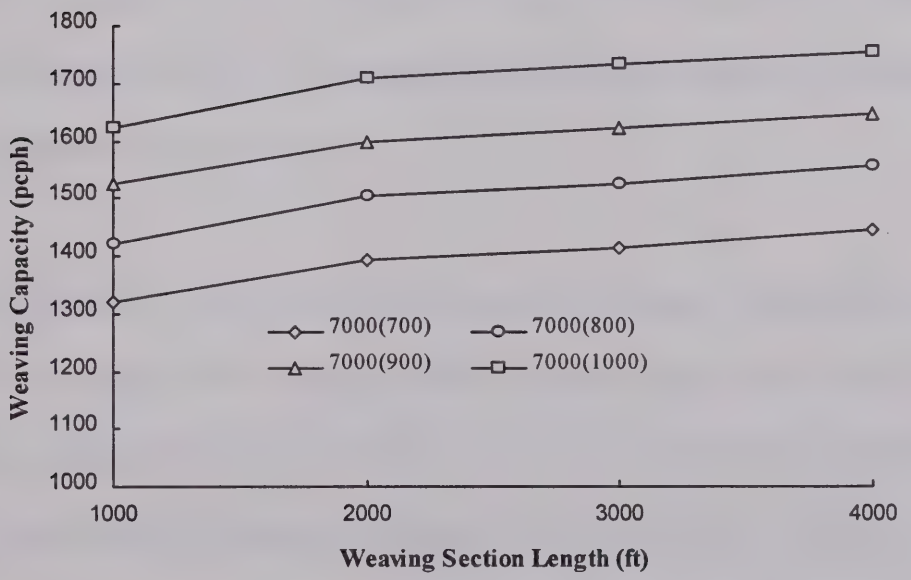
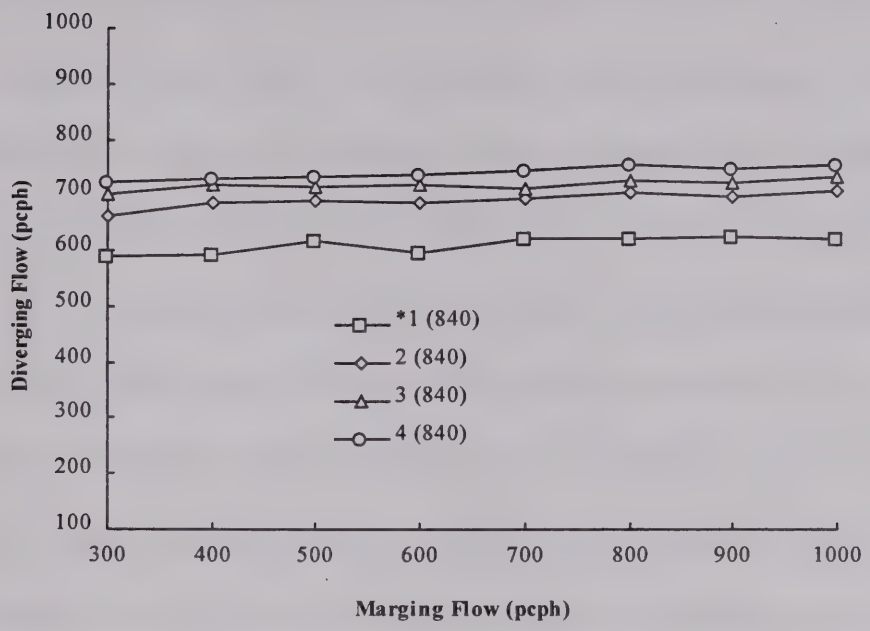


**Figure 4-6 Diverging Flow vs. Marging Flow**




*ALL in thousands (off-ramp flow)

the weaving section capacity is relatively not sensitive to the ALL (See Table 4-3). Additionally, we found that as the weaving flow increases as the entry flow is fixed, the observed weaving section capacity increases. These results are surprisingly different from what we anticipated (See Table 4-3).

The 1994-HCM suggests that if the weaving section length is less than 2000 feet the section can be analyzed by weaving models. In another word, if the weaving section length is greater than 2000 feet, the section is recommended to be separated into one on-ramp and one off-ramp junctions and analyzed by ramp junction models. However, simulation results show that the weaving section capacity is still influenced by the

weaving section length when it is beyond 2000 feet (Table 4-3) depending on various traffic and geometric conditions, and local driving behavior. It is hard to identify the critical section length (beyond which the weaving flows have no influence on weaving section capacity) from Figure 4-2. However, results obtained from CORSIM can definitely be used to assist HCM in various analysis, such as the impact analysis of the ALL beyond 2000 ft and the location of warning sign. After comparing the results derived from both CORSIM and HCM, Figure 4-3 shows that when the ALL is fixed (e.g., 2000 feet), the weaving capacity observed from the simulation results is around 1700 pcph while the specified weaving flow is 1800 pcph. This value is different from the maximum weaving flow addressed in 1994-HCM, 1800 pcph. The difference between the outputs from CORSIM and HCM cannot help to indicate which model is better but suggest that using simulation models can help to complement the limitations or fill gaps in HCM.

Results generated by CORSIM for Scenario 2 are summarized in Tables 4-6 and 4-7, and shown in Figures 4-4 through 4-6 . Conclusions are made as follows:

1. The weaving section capacity increases as either the merging flow or the ALL increases (Fig. 4-4).
2. Similar to Scenario 1, as the auxiliary lane length is fixed as 2000 feet, the weaving capacity is around 1700 pcph while the input weaving flow is 1840 pcph (Figure 4-5). However, the capacity may increase if the total weaving flow exceeds 1840 pcph.

3. The observed diverging flow increases when the merging flow increases (Figure 4-6). Since the increased merging flow interrupting the through flow, diverging vehicles may easily find safer gaps. For future investigation of this issue, simulation of weaving sections with various numbers of lanes on the mainline as well as the field investigation are required.

## 4.4 Speeds in Weaving Areas

In Chapter 4 of the 1994-HCM, twelve equations were developed for estimating speeds of weaving and non-weaving flows under various conditions. The general model of those equations is

$$S_W \text{ or } S_{NW} = 15 + \frac{50}{1 + a(1+VR)^b (v/N)^c / L^d} \qquad (4\text{-}1)$$

where a, b, c, and d are constants. In Eq. 4-1, $S_W$ and $S_{NW}$ represent the average speeds for both weaving and non-weaving flows, respectively. Variable include VR, v, N, and L representing the volume ratio, the total flow rate in the weaving area, total number of lanes in the weaving area, and the length of weaving area (feet), respectively.

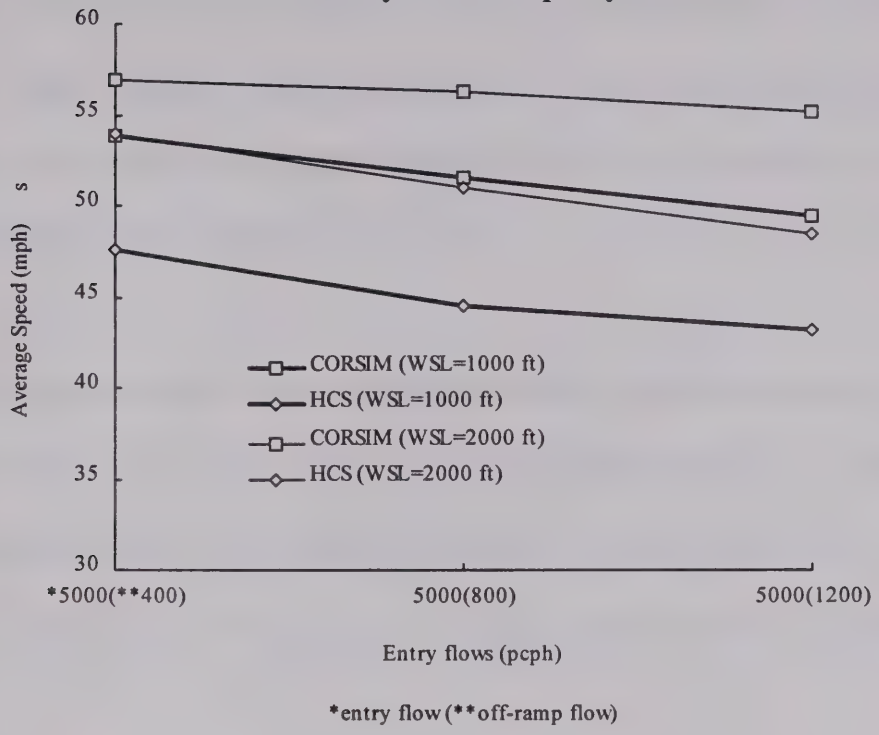
However, Eq. 4-1 did not consider two important factors including (1) the free-flow speed or the speed limit of the weaving area and (2) the weaving ratio (the diverging flow to the merging flow) of the analyzed area. These factors may affect weaving and non-weaving speeds and the level of service of the weaving area. For example, the

**Table 4-8 Average Speeds in Weaving Areas from CORSIM and HCS**

| Input Flow | Average Speed | | Average Speed | |
|---|---|---|---|---|
| | CORSIM (ALL=1000 ft) | HCS (ALL=1000 ft) | CORSIM (ALL=2000 ft) | HCS (ALL=2000 ft) |
| *5000(400) | 53.83 | 47.57 | 56.95 | 53.93 |
| 5000(800) | 51.51 | 44.5 | 56.25 | 51 |
| 5000(1200) | 49.46 | 43.14 | 55.2 | 48.43 |
| 5714(400) | 48.19 | 46.73 | 53.58 | 53.89 |
| 5714(800) | 45.84 | 44.45 | 52.03 | 51.45 |
| 5714(1200) | 42.98 | 43.58 | 51.07 | 48.43 |
| 6667(400) | 37.49 | 45.9 | 36.85 | 50.45 |
| 6667(800) | 34.06 | 43.65 | 34.19 | 50.65 |
| 6667(1200) | 33.19 | 43.78 | 34.04 | 48.52 |

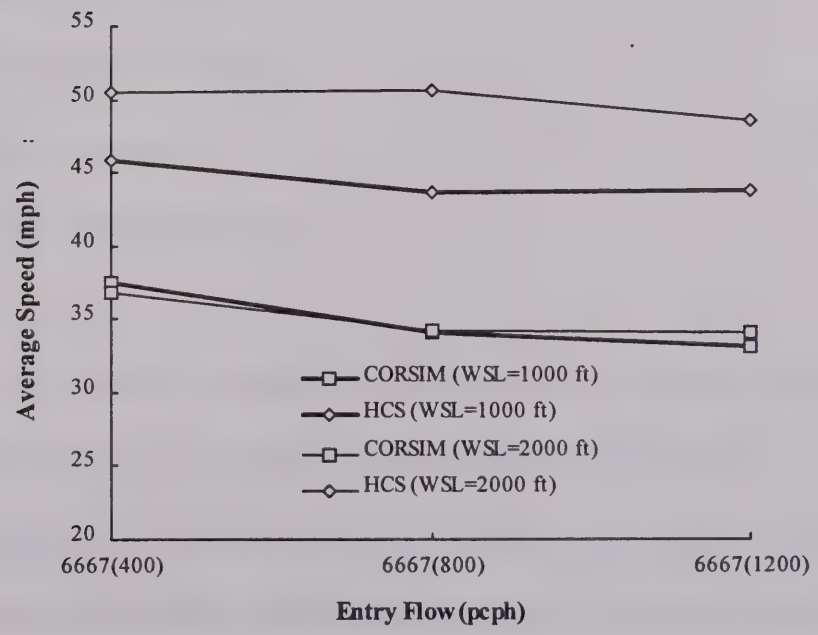
*input flow (diverging flow)

**Figure 4-7 Average Speeds (CORSIM vs. HCS) as Entry Flow < Capacity**




*entry flow (**off-ramp flow)

**Figure 4-8 Average Speeds (CORSIM vs. HCS) as Entry Flow >= Capacity**



average speeds in weaving area differ when the diverging flow of 800 pcph with the merging flow of 600 pcph, and diverging weaving flow of 600 pcph with the merging flow of 800 pcph, although total weaving flow are the same (both are 1400 pcph).However, the results of both cases obtained from HCM are identical but different when they are analyzed by CORSIM.

The estimated average speeds obtained from HCS for both weaving and non-weaving flows can not be generated from the current CORSIM model. To compare the speed information obtained from CORSIM and HCS, the average speed ($S_{avg}$) can be obtained by substituting the weaving and non-weaving speeds derived from HCM into Eq. 4-2.

$$S_{avg} = \frac{S_W F_W + S_{NW} F_{NW}}{F_W + F_{NW}} \qquad (4\text{-}2)$$

where $S_W$ : the weaving speed

$S_{NW}$ : the non-weaving speed

$F_W$ : the weaving flow

$F_{NW}$ : the non-weaving flow

Since the maximum weaving section length (WSL) is limited to 2000 ft in HCM (if WSL > 2000 feet, it is suggested to be analyzed as two isolated ramp junctions), comparative analysis of the average speeds is conducted only considering various ALLs (WSL defined in HCM is little higher than ALL) equal to or less than 2000 feet. Table 4-

8 summarizes the average speeds obtained from CORSIM and HCS and shown in Figures 4-7 and 4-8. The average speeds obtained from both softwares are inconsistent while considering the same geometric and traffic conditions. CORSIM demonstrates a higher average speed than that derived from HCS when demand is less than capacity (i.e., 5000 pcph and 5714 pcph). On the other hand, a lower average speed was found from CORSIM than that derived form HCS as the entry flow exceeds capacity (e.g., 6667 pcph and 7000 pcph). However, the difference between the average speeds produced by CORSIM and HCS may vary for the same network because the values of other parameters (e.g., the CFSF in CORSIM and the driver population factor ($f_p$) in HCS) are not calibrated. Such inconsistencies can be overcome by performing calibration in both the CORSIM and the HCM models.

# CHAPTER 5

# SUMMARY

## 5.1 Conclusion

Factors most significantly affect freeway capacity on basic freeway section, including the car following sensitivity factor, the grade section length and its percentage, and the truck percentage. In analyzing ramp junctions and weaving areas, we found that the capacity further affected by the mainline and merging/diverging flows, the warning sign location, and the auxiliary lane length (ALL). In addition, factors affecting freeway capacity for basic freeway sections, ramp-junctions, and weaving areas have been conducted through exhaustively comparative and sensitivity analysis. Guidelines for approximating freeway capacity and determining the critical ALL at ramp junctions and in weaving areas are also developed.

We found that the CFSF, the grade, and the truck volume affect freeway capacity in different degrees. The relations among these factors and freeway capacity were found from the corresponding figures and tables shown in each chapter. For example, the relations between CFSF and freeway capacity identified in this study can be used as a reference and help users to calibrate CFSFs for the analyzed networks.

While considering the default CFSF in CORSIM, $E_T$ values at various truck percentages and grade section lengths are developed. The $E_T$ values derived from simulation results are partially different from that suggested by HCM, specially, as the grade percentage is high (e.g., 4% and 6% grade). Furthermore, the relations between the ALL and the approximated capacity (from CORSIM results) of ramp junctions and weaving areas provide valuable information to traffic engineers for determining cost effective auxiliary lanes.

Although figures developed for this study did not cover diversified geometries and factors, the proposed procedure and method can be used to either explore extensive relations among various factors and capacity in the future or validate the simulation model. CORSIM can not model every special situations (e.g., lane width and lateral clearance), but is an essential and a valuable tool to evaluate freeway operational performance and approximate capacity.

Speed is one of the most important elements in evaluating level of service in weaving areas. With the increase of the weaving flow, turbulence in a weaving area increases and consequently the average speed reduces. In a test run by using CORSIM, we found that variations of merging and diverging flow (for the constant total weaving flow) will affect the average weaving and non-weaving speeds in weaving areas. However, under such situations, the average speeds for both weaving and non-weaving flows generated by HCM remain constant. This issue need to be further tested and validated as well.

Simulation results showed that the critical ALL in ramp junctions exceeds 1500 feet and the critical ALL in weaving area exceeds 2000 ft still affecting the capacities in both areas. Although HCM has limitations because of limited database, CORSIM can assist HCM to overcome such deficiencies. In addition, it is also essential to validate models suggested in HCM and the vehicle lane changing logic in CORSIM through extensive field observations.

## 5.2 Future Research

We suggest that the following studies should be conducted in the future:

- Investigating inconsistencies of estimated speeds obtained from CORSIM and HCS.
- Investigating the impact of the weaving flow on the weaving section capacity when the weaving section length is greater than 2000 ft.
- Investigating both CORSIM and HCM to improve the accuracy of the suggested $E_T$ values.
- Investigating the critical ALLs for various ramp junctions and weaving areas.
- Investigating whether capacity is influenced by the number of lanes on freeways.
- Investigating the impact of the warning sign location on the freeway capacity under various geometric conditions, such as ramp junctions, weaving areas, and workzones.

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